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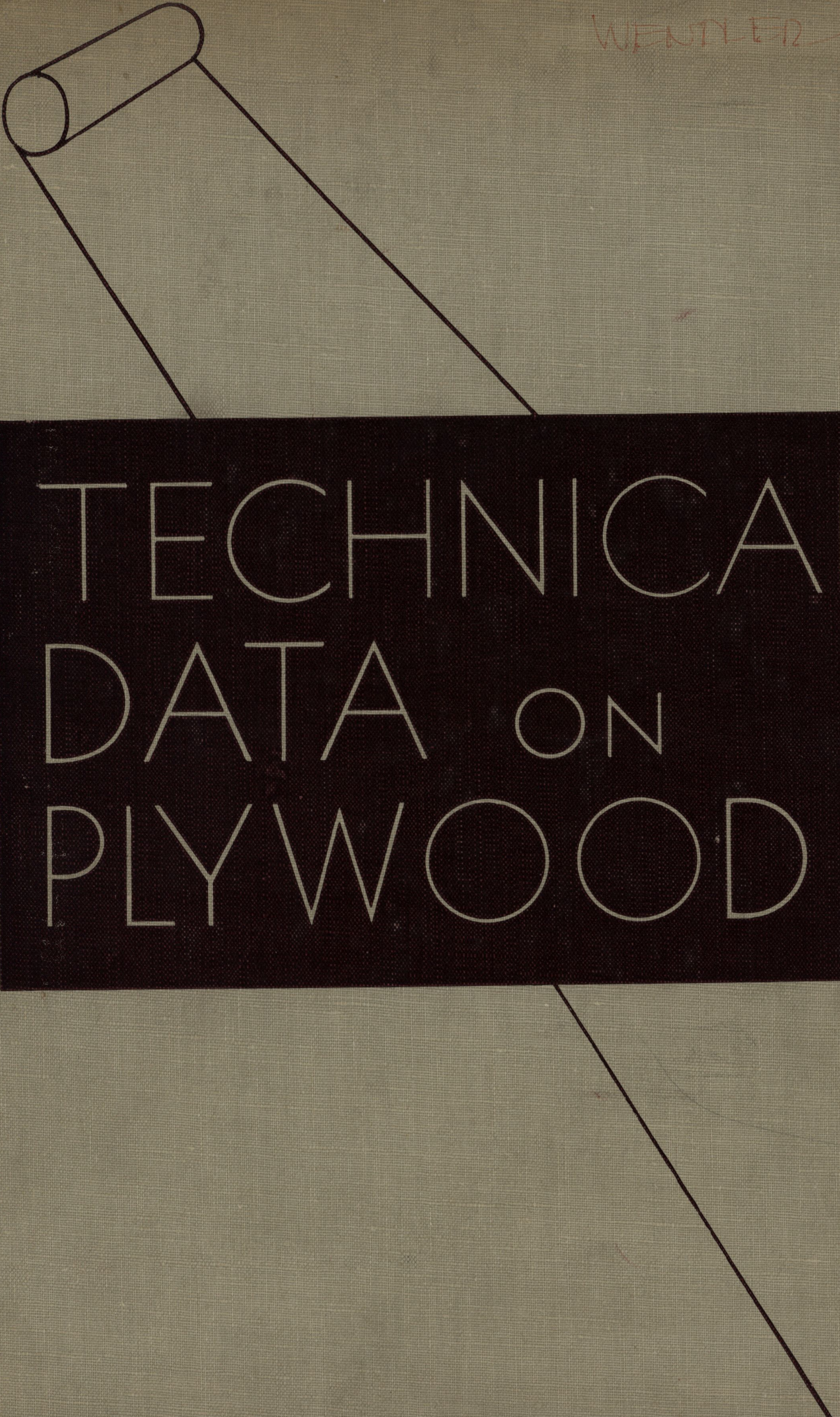

TECHNICAL DATA ON PLYWOOD

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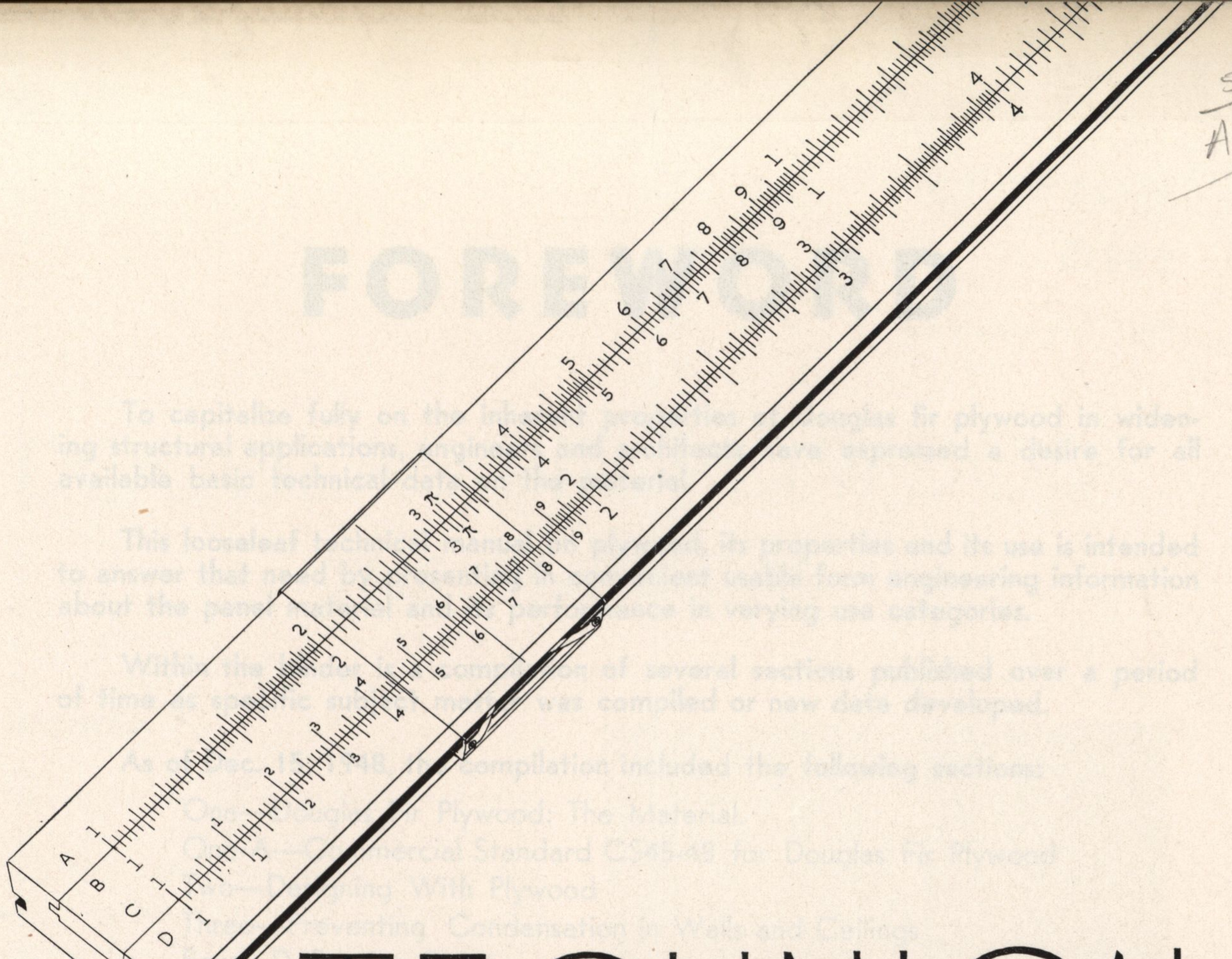

TECHNICAL DATA ON DOUGLAS FIR PLYWOOD FOR ENGINEERS AND ARCHITECTS

FOREWORD

To capitalize fully on the inherent properties of Douglas fir plywood in widening structural applications, engineers and architects have expressed a desire for all available basic technical data on the material.

This looseleaf technical manual on plywood, its properties and its use is intended to answer that need by presenting in convenient usable form engineering information about the panel material and its performance in varying use categories.

Within the binder is a compilation of several sections published over a period of time as specific subject matter was compiled or new data developed.

As of Dec. 15, 1948, the compilation included the following sections:

One—Douglas Fir Plywood: The Material.
One-A—Commercial Standard CS45-48 for Douglas Fir Plywood
Two—Designing With Plywood
Three—Preventing Condensation in Walls and Ceilings
Four—Deflection Charts
Five—Insulating With Plywood
Six—Lateral Bearing Strength of Nailed Plywood Joints
Seven—Form Factors of I and Box Beams with Plywood Webs
Eight—The Design of Flat Panels With Stressed Covers
Nine—The Design of Built-Up Beams with Plywood Webs

Additional sections will be issued as wider technical data are amassed from test observations and use experience. Such new sections will be mailed directly to the holders of the handbook; further, revisions of, or supplements to, existing sections will be mailed in order to assure that this basic technical information is current at all times.

It is suggested, therefore, that this binder be kept readily available and that subsequent chapters be immediately placed in the binder as soon as they are received.

Comments on the data contained herein and suggestions for further chapter material are invited.

DOUGLAS FIR PLYWOOD ASSOCIATION
Tacoma 2, Washington

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SECTION ONE

(Revised November 1, 1948)

DOUGLAS FIR PLYWOOD:
THE MATERIAL

DEFINITION:

Douglas fir plywood is an engineered wood board or panel, consisting of an odd number of Douglas fir veneer sheets placed crosswise and bonded together, under hydraulic pressure, with either water-resistant or waterproof adhesives, each stronger than the wood itself.

An odd number of veneer sheets are used to give a balanced, symmetrical panel construction, thus minimizing warping, cupping, and twisting.

The better veneers are used for faces of panels while lower grade stock is placed in the cores and cross-bands.

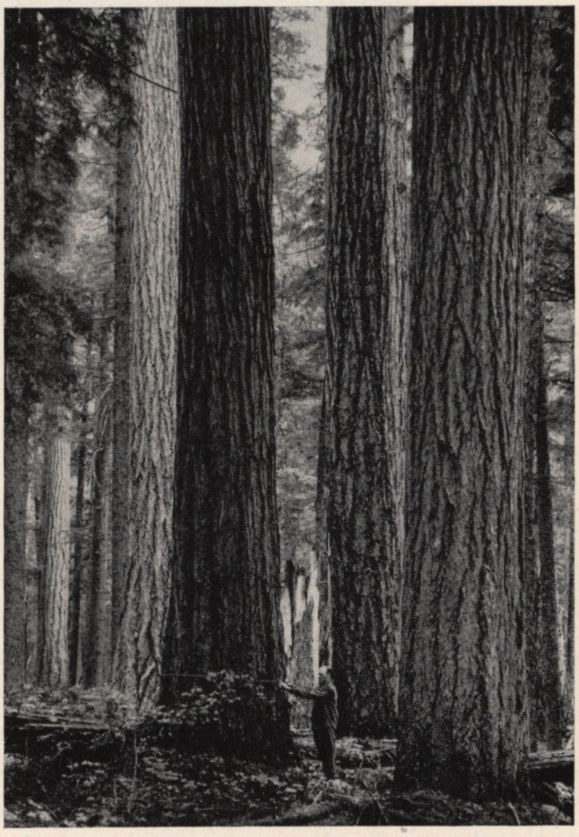
DESCRIPTION:

Douglas fir plywood is manufactured in wide, long panels in standard sizes up to 48″ wide and 96″ long, and in various standard thicknesses from 3/16" to 1 1/8".

Panels five (5) feet wide and ten to twelve feet long are also commonly produced on special order, and one mill can produce panels sixteen (16) feet long, by using face veneers cut from a sixteen (16) foot log.

In addition, several plants can form large sheets 8 feet long (or wide) to 50 feet in the other direction, by scarf-jointing 4′ x 8′ panels together along the 8′ dimension.

Two types of Douglas fir plywood are made, i. e., the Exterior Type, with a waterproof phenolic resin adhesive, and the Interior Type. Suitable tests are prescribed for each type. (See page 4.)



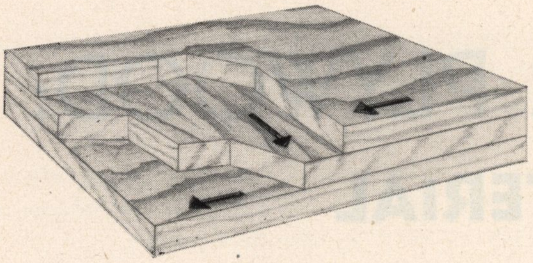
U. S. Forest Service Photo

Characteristic stand of Douglas fir.

Each type, in turn, is produced in several standard grades, which are based on appearance and utility. These grades have been developed chiefly as a result of consumer demand, as indicated by many years experience, with grade limits fixed by the physical characteristics of Douglas fir logs and the veneers which it is possible to peel from them.

PHYSICAL PROPERTIES:

The construction of a plywood panel by cross-laminating veneers and gluing them together capitalizes on the more desirable physical properties of wood. The strength of wood along the grain is many times greater than across the grain. Moreover, wood has virtually no expansion or contraction along the grain, even with large changes in moisture content. Further, wood has no cleavage plane across the grain and consequently cannot split in that direction.



Sketch showing how each layer of veneer has its grain running perpendicular to that in adjacent plies.

In plywood, since adjacent plies are placed crosswise to each other, longitudinal grain is provided both lengthwise and crosswise of the panel. Consequently plywood functions effectively as a structural diaphragm or as a bracing, and also where plate action, with all edges supported, is involved. The cross lamination in plywood tends to equalize the properties in each direction and to minimize, if not entirely eliminate, certain weaknesses.

Shrinkage, for example, is reduced in plywood so as to be negligible in many services.

Splitting is impossible in any direction.

Bending strength and stiffness, although reduced somewhat lengthwise of the panel, are greatly increased crosswise, the amount varying with the number and thickness of plies. The greater the number of plies, of course, the nearer the equalization of strength in the two directions.

Douglas fir plywood, in addition to rendering a wall airtight, has favorable thermal insulation properties. It rates well acoustically, both in absorbing sound and in reducing sound transmission.

It is easily worked, cut to various sizes and shapes, and may be bent to fairly sharp radii. It may be finished in a great variety of ways.

An important advantage in construction and in many industrial uses is the speed with which plywood may be installed.

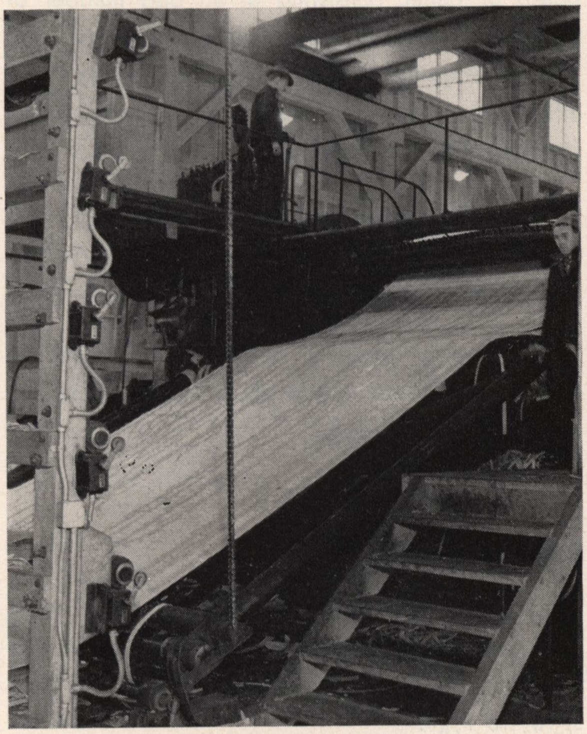
MANUFACTURE:

Douglas fir trees, from virgin forests on the western slopes of the Cascades and the coastal plains of the Pacific furnish the giant "peeler" logs that are used in making Douglas fir plywood.

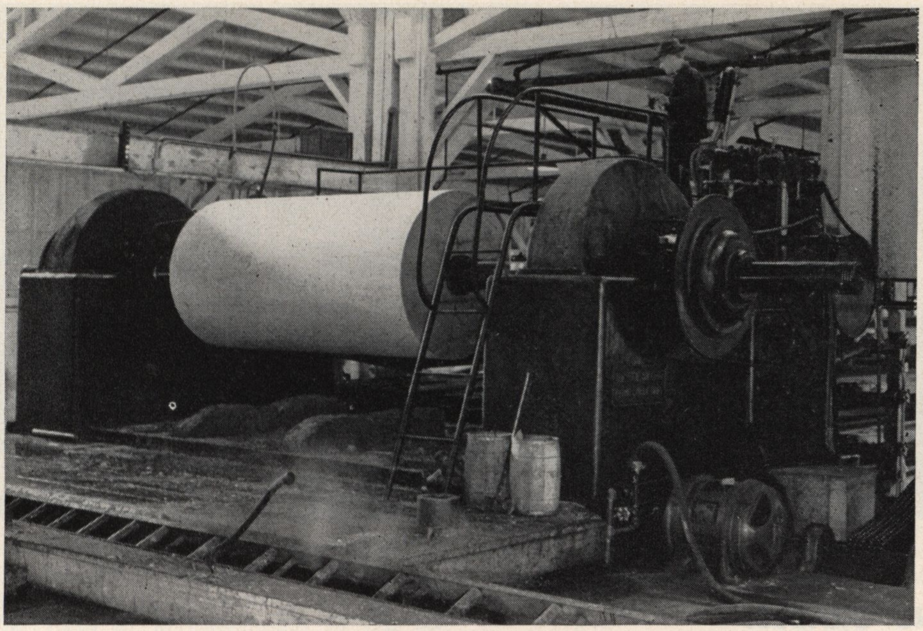
The transformation of these logs into either the Interior or the Exterior Type of plywood, although requiring skill, precision, and modern equipment, nevertheless consists basically of a few fairly simple steps.

Selected peeler logs, averaging 3 to 4 feet in diameter, are first cut into desired lengths or bolts, usually about 8 feet long.

The bolt is placed in a giant lathe and rotated against a long knife which peels the wood in long, continuous, thin sheets known as veneer. The veneer is conveyed to clippers which cut it to desired widths, after which it is run through dryers and dried to about 2 or 3 percent moisture content. After careful grading, the veneer goes to the glue spreaders where in the manufacture of cold-pressed Interior Type plywood it is assembled into panel form in stacks about 5 feet high. These stacks are then rolled onto the bed of a huge hydraulic press. After pressure of about 150 lbs. per square inch is applied, the stacks are clamped to maintain pressure for a number of hours while



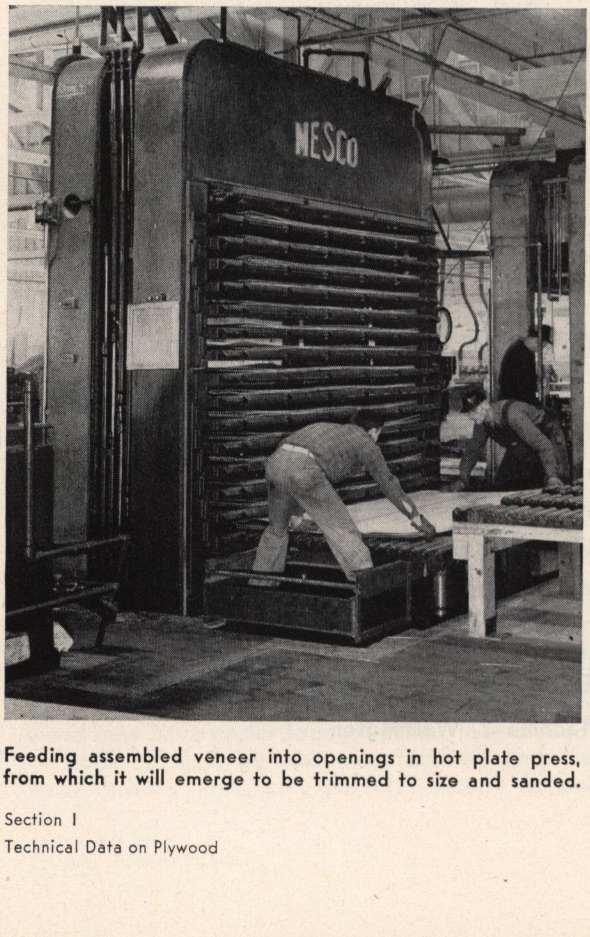
Long sheets of veneer are peeled from the large bolts of Douglas fir, and later clipped to desired widths.



Typical Douglas fir bolt in lathe ready for peeling into veneer.

the glue is setting, and removed from the press. Later the newly formed plywood panels are cut and trimmed to exact size, and run through sanding machines from which they emerge as finished plywood. A final grading and the panels are ready for shipment.

During the past few years most of the Interior Type of Douglas fir plywood has also been produced by a hot-plate process. The method is quite similar to the Exterior process, except for the type of adhesive used.



Feeding assembled veneer into openings in hot plate press, from which it will emerge to be trimmed to size and sanded.

"Exterior" Type Douglas fir plywood is made by the hot-press process, using phenolic resin adhesives that are pressed and completely set within a few minutes by means of large hot-plate presses. These open up vertically like giant accordions, with a number of parallel, thick, steel plates, one above the other, which are heated by steam. The veneer sheets, peeled, clipped, and dried much the same as for cold-pressed plywood, are glued and assembled into panels which are placed in pairs, or singly if the panels are thick, into the several openings between the press plates. The press then closes up, providing pressure and heat needed to change the phenolic resin adhesive into an insoluble plastic, and welding the cross-laminated veneers into large, rigid panel units, suitable for permanent weather exposure.

U. S. COMMERCIAL STANDARD FOR DOUGLAS FIR PLYWOOD:

On August 17, 1932, manufacturers, distributors, and users of Douglas fir plywood approved the adoption of standard grading rules that were accepted by the trade and promulgated as "Douglas Fir Plywood, Commercial Standard CS45-33."

Since then, this standard has been revised periodically in order to meet changing consumer demands and manufacturing conditions. The latest edition, effective November 1, 1948, is known as U. S. Commercial Standard CS45-48.

Glue testing scenes in laboratory of Douglas Fir Plywood Association, Tacoma, Washington.

TYPES AND GRADES:

This standard provides for the two basic types of Douglas fir plywood, i. e., Exterior and Interior; it also provides the performance tests which each type must be capable of passing.

The distinguishing feature between "Exterior" panels and "Interior" panels lies in the durability of the adhesives employed.

"Exterior" Type Douglas fir plywood is made with a hot-pressed synthetic resin adhesive which is permanent and insoluble under practically any exposure condition, including boiling. It is designed for permanent exterior exposure.

"Interior" Type Douglas fir plywood is made usually with protein glues having a soya bean base, in either a hot or cold press, or with extended phenolic resins in a hot press. Such plywood is designed for permanent interior service and also to withstand temporary weather exposure or wetting during construction when used for subfloors, sheathing of buildings and similar purposes.

In general, this Interior plywood is suitable for service where the moisture content of wood or plywood will be less than 20%, or, in other words, wherever there is no danger from mold, fungus growth, or decay.

Each type is manufactured in several "appearance" grades, established to fit a wide variety of uses. The grade of a panel depends chiefly upon the quality of veneer used in the face and back, although certain other limitations are also imposed.

Under the new standard, all plywood is built up out of one or more of the four standard veneer grades, i. e.,

(1) Grade A (Sound)—a sound, tight veneer free from knots or open defects; suitable for painting.

(2) Grade B (Solid)—may contain same characteristics as Grade A, and also plugs, solid knots, and minor sanding defects.

(3) Grade C (Exterior Back)—the type of veneer used on back of EXT-DFPA•Plyshield; knot holes limited to 1 inch maximum; splits limited to 3/16 inch.

(4) Grade D (Utility)—may contain solid knots, knot holes not more than 2½ inches maximum dimension, plugs and so on. (For complete description of grades see Section 1-A, Commercial Standard CS45-48)

Interior Type plywood has veneer of Grade D quality or better for its crossbands.

Exterior Type has its inner plies of Grade C (Exterior Back) veneer quality or better.

Plywood produced by mills of the Douglas Fir Plywood Association to conform with U. S. Commercial Standard CS45-48 quality carries a copyrighted grade mark either branded or stamped on edge or back. This permits easy identification and assures consumer of quality specified.

DOUGLAS FIR PLYWOOD ASSOCIATION

Tacoma 2, Washington

Note: This revision issued
Feb. 1, 1955, **replaces**
Section One-A revised as of
Nov. 1, 1946.

SECTION
ONE-A REVISED

February 1, 1955

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DOUGLAS FIR PLYWOOD COMMERCIAL STANDARD CS45-55

(Ninth Edition—Supersedes CS45-48)

1. PURPOSE

1.1 Because of the extended application of Douglas fir plywood to a large number of new uses, the standard grading rules given herein are offered as a universal basis of understanding in the industry. General adoption and use of this standard will facilitate procurement of the proper grade of material and the proper type as to moisture resistance for its varied uses, and provide a better understanding between buyer and seller. Architects, engineers, contractors, industrial users, and home owners will thus be able to specify their needs from nationally accepted grading standards.

2. SCOPE

2.1 These rules cover seven grades and two special items of Interior type and seven grades of Exterior type Douglas fir plywood, which is a laminated board suitable for paneling, sheathing, subflooring, exterior siding, concrete forms, cabinet work, and many other structural and industrial uses. The standard includes tests, standard sizes, size tolerances, reinspection rules, grade-marking provisions, and nomenclature and definitions. It also provides descriptions and rules governing the manufacture of overlaid plywood.

3. DEFINITION

3.1 Douglas fir plywood is a built-up board of laminated veneers in which the grain of each piece is at right angles to the one adjacent to it. The kiln-dried veneer is united under high pressure with a bonding agent, making the joints as strong as or stronger than the wood itself. The alternating direction of the grain of each contiguous layer of wood equalizes the strains, and in this way minimizes shrinkage and warping of the product and prevents splitting. Overlaid plywood is produced in a like manner with the special facings added.

4. REQUIREMENTS

4.1 **Workmanship.**—Unless otherwise specified, plywood shall be sanded on two sides to meet requirements of veneer as set forth in paragraph 4.4.4. When specified rough or unsanded, plywood may have paper tape on either face or back, or both. It shall be well manufactured and free from blisters, laps, and defects, except as permitted in the specific rules for the various grades. Exposed veneer on both sides of panel shall have bark or tight surface out. Plies directly under surfaces of overlaid panels are not considered exposed veneers.

4.2 **Bonding.**—The entire area of each contacting surface of the plywood shall be bonded in an approved manner with material best adapted to each use classification. No tape shall be used in any glue line.

4.3 **Loading or Packing.**—The plywood shall be securely loaded or packed to insure delivery in a clean and serviceable condition.

4.4 **Types of Plywood.**—Douglas fir plywood is made in two types, Interior (Int.) and Exterior (Ext.), with the type referring to the moisture resistance of the adhesives bonding the plies together. Within each type there are several grades, which are established by the quality of the veneer on both faces of the panel as hereinafter defined. The grade descriptions set forth the minimum requirements, and, therefore, the majority of panels in any shipment will exceed the specification given.

4.4.1 **Moisture Content.**—Moisture content of panels at time of shipment from mill shall not exceed 18% of dry weight as determined by oven-dry test.

4.4.2 **Veneers.**—Veneers shall be 1/12 in. or more thick before sanding in panels 1/4 in. and thicker, except that veneers 1/16 in. thick before sanding may be used in 5-ply, 3/8 in. thick Exterior type panels. The veneer in any particular classification, as set forth in paragraph 4.4.4, shall not contain any defect larger than those permitted specifically therein, or that will significantly impair either the strength or the serviceability of the panel. Sound, firm stain shall not be considered a defect.

4.4.3 **Ring Count.**—A minimum of six annular rings per inch, as measured in block at time of peeling, shall be required for both faces of all grades.

4.4.4 **Veneer Classifications.**—All veneers used in the different plywood grades shall be one of the following (grade A being the best of the four veneers):

4.4.4.1 **Grade A** veneer shall be of one or more pieces of firm, smoothly cut veneer. When of more than one piece, the pieces shall be well joined. The veneer shall be free from knots, open splits, pitch pockets, and other open defects. Pitch streaks averaging not more than 3/8 in. in width and blending with color of wood, discolorations, sapwood, shims, and neatly made patches shall be admitted, but not more than 18 veneer patches shall occur in any 4 ft. x 8 ft. **A** face, with proportionate limits for other sizes of panels. Shims may not be used over or around patches. Any multiple repair in a panel shall be limited to two patches. All patches and repairs must run parallel to the grain. However, approved plastic fillers may be used to fill small cracks or checks not more than 1/32 in. wide; to fill small splits or openings up to 1/16 in. wide, if not exceeding 2 in. in length, and also to fill small chipped areas or openings not more than 1/8 in. wide by 1/4 in. long. This grade shall present a smooth surface suitable for painting.

4.4.4.2 **Grade B** veneer shall present a solid surface, free from open defects except as noted, but in addition to characteristics admitted in Grade **A**, veneer shall admit also neatly made circular plugs, as well as synthetic plugs that present solid, level, hard surfaces, knots up to 1 in. if both sound and tight, splits not wider than 1/32 in., slightly rough but not torn grain, and other minor sanding and patching defects, including sander skips not exceeding 5% of panel area. Tiny vertical holes not exceeding 1/16 in. in diameter caused by ambrosia beetles are admissible if not exceeding an average of one per square foot in number; also admissible are horizontal or surface tunnels, which shall be limited to 1/16 in. across, 1 in. in length, and to 12 in number in a 4 ft. x 8 ft. panel, or proportionately in panels of other dimensions.

4.4.4.3 **Grade C** veneer may contain knotholes not larger than 1 in. in least dimension; open pitch pockets not wider than 1 in.; splits not wider than 3/16 in. that taper to a point; worm or borer holes not more than 5/8 in. wide or 1 1/2 in. long; knots if tight and not more than 1 1/2 in. in least dimension; and plugs, patches, shims, sanding defects, and other characteristics in number and size that will not impair the serviceability of the panel.

4.4.4.4 **Grade D** veneer (may be used only in Interior type panels) shall contain no knotholes greater than 2 1/2 in. in maximum dimension, no pitch pockets more than 2 in. wide by 4 in. long, or of equivalent area if of lesser width, and no splits wider than 1/2 in. Splits 1/2 in. wide at widest point may be one-fourth-panel length; those not more than 1/4 in. wide at widest point may be half-panel length; and those not more than 3/16 in. wide may be full-panel length, but all splits shall taper to a point at one end. Any number of plugs, patches, shims, worm or borer holes, sanding defects, and other characteristics are permitted provided they do not seriously impair the strength or serviceability of the panel.

4.4.5 **Overlays.**—Overlaid plywood is Douglas fir plywood to which has been added resin-impregnated fiber faces on one or both sides. It is made in two types, High Density and Medium Density, with the type referring to the surfacing material as hereinafter defined. The resin-impregnated faces are permanently fused to the base panel under heat and pressure. Although designed for either exterior or interior service, all overlaid plywood is made in the Exterior type. This refers to the adhesive bond between plies, between the overlay surface and the base panel, and to the durability of the surface itself.

4.4.5.1 **High Density Type.**—The surfacing on the finished product shall be hard, smooth, and of such character that further finishing by paint or varnish is not required. It shall consist of a cellulose-fiber sheet or sheets, in which not less than 40 percent by weight of the laminate shall be a thermo-setting resin of the phenol or melamine type. The resin-impregnated material shall be not less than 0.009 in. thick and shall weigh not less than 60 pounds per 1,000 square feet of single face, including both resin and fiber. The resin impregnation shall be sufficient to attach the surfacing material to the plywood. This bond shall be equal in performance to the glue lines between the sheets of veneer which make up the plywood. The overlay face usually comes in natural translucent color, but certain other colors are available or may be used by manufacturers for identification.

4.4.5.2 **Medium Density Type.**—The resin-impregnated facing on the finished product shall present a smooth, uniform surface suitable for high-quality paint finishes. It shall consist of a cellulose-fiber sheet in which not less than 20 percent by weight of the laminate shall be a thermosetting resin of the phenol or melamine type. The resin-impregnated material shall be not less than 0.012 in. thick and shall weigh not less than 65 pounds per 1,000 square feet of single face, including both resin and fiber. An integral phenolic resin glue line shall be applied to one surface of the facing material to bond it to the plywood. This bond shall be equal in performance to the glue lines between the sheets of veneer which make up the plywood. The overlay face shall be a solid color. Some evidence of the underlying grain may appear, but, compared to the nature of the "high density" surface, there shall be no consistent show-through.

4.5 **Interior Type Plywood.**—This type of plywood has a high degree of moisture resistance and is suitable for constructions where its application requires that it shall retain its original form and practically all its strength when occasionally subjected to a thorough wetting and subsequent normal drying; it is also suitable for constructions where subjected to occasional deposits of moisture by condensation through walls or leakage, or from other sources. All veneer used in Interior type shall be of Douglas fir, except that Western hemlock, Sitka spruce, noble fir, commercial white fir, Alaska cedar, Port Orford cedar, California redwood, ponderosa pine, sugar pine, Idaho white pine, and Western larch may be used for inner plies only, in Interior type grades **A-A, A-B, A-D, B-D,** and the special "natural finish" items (see table 1). Plywood of this type shall meet the test requirements set forth in paragraphs 5.2 and 5.4.1. This type is available in the grades given in table 1.

NOTE: Interior Sheathing, Underlayment, and Concrete Form grades shall be made with an adhesive possessing a mold resistance equivalent to that created by adding 5 pounds of pentachlorophenol, or its sodium salt, per 100 pounds of dry glue base to plain protein glues.

Table 1. Interior Type Grades—Minimum Quality of Veneers

Grade	Face	Back	Inner Plies	Additional Limitations[1]
A-A, Int.	A	A	D	Sanded 2 sides.
A-B, Int.	A	B	D	Sanded 2 sides.
A-D, Int.	A	D	D	Sanded 2 sides.
B-D, Int.	B	D	D	Sanded 2 sides.
C (Repaired)-D, Int. (Underlayment, Int.)	C (Repaired)[2]	D	C[3] or D	Sanded 2 sides.
C-D, Int. (Sheathing, Int.)	C	D	D	Unsanded grade. No belt sanding permissible.
B-B, Int. (Concrete Form, Int.).[4]	B	B	C (All Inner Plies).	Edge-sealed and, unless otherwise specified, mill-oiled. Sanded 2 sides.
N-D, Int. (Natural Finish One Side).[5]	Special[5]	D	D	Sanded 2 sides.
N-N, Int. (Natural Finish Two Sides).[6]	Special[6]	Special[6]	B[7]	Sanded 2 sides.

[1] See also paragraphs 4.4 and 4.5.

[2] Face may contain knotholes, worm or borer holes, and other open defects not larger than 1/4 by 1/2 in., sound and tight knots up to 1 1/2 in. in greatest dimension, splits up to 1/16 in. wide, ruptured and torn grain, sander skips up to 5 percent of panel area, pitch pockets if solid and tight, plugs, patches, and shims.

[3] Veneer immediately adjacent to face shall be **C** or better.

[4] B-B, Int., unless ordered as a Concrete Form grade or edge-sealed, may be furnished with **D** inner plies.

[5] A special order "one side Natural Finish" item, not generally available in stock, intended primarily for paneling and wainscoting, generally only in 1/4 in. thickness. Available only from certain mills. The face shall consist of smoothly cut veneer of 100 percent heartwood free from knots, splits, pitch pockets, and other open defects; not more than 3 pieces of veneer shall be used, and they shall be well matched as to color and grain. Faces shall be of a yellow or pinkish color without stain. Two shims, neither longer than 6 in., that occur only at the ends of panels, and not more than 4 inconspicuous well-matched small patches not to exceed 3/8 in. wide by 2 1/2 in. long shall be admitted. All repairs and all veneer joints shall be parallel to the edges of the panel. No overlapping of repairs is permitted. In all other respects the panel shall conform to an Interior type A-D panel.

[6] Also a special order "two sides Natural Finish" item, intended primarily for cabinet work, generally only in 3/4 in. thickness. Available only from certain mills. Each face shall consist of veneer equivalent to face described in footnote 5.

[7] All inner plies shall consist of **B** veneer with crossbands jointed.

4.6 **Exterior Type Plywood.**—This type represents the ultimate in moisture resistance—a plywood that will retain its original form and strength when repeatedly wet and dried and otherwise subjected to the elements, and which is suitable for permanent exterior use. It shall be free from both core gaps and core voids that impair the strength or serviceability of the panel. All repatches and shims shall be set wtih adhesives meeting performance standards for Exterior plywood. All veneer used in Exterior type panels shall be of Douglas fir and of **C** grade as defined in pargraph 4.4.4 or better. All Exterior panels shall be so designated by a distinctive symbol "Ext" branded or stamped on the edge of each panel. Plywood of this type shall meet the test requirements set forth in paragraphs 5.3, 5.4.2, and 5.4.3. This type is available in the grades given in table 2.

Table 2. Exterior Type Grades—Minimum Quality of Veneers

Grade	Face	Back	Inner Plies	Additional Limitations[1]
A-A, Ext.	A	A	C	Sanded 2 sides.
A-B, Ext.	A	B	C	Sanded 2 sides.
A-C, Ext.	A	C	C	Sanded 2 sides.
B-C, Ext.	B	C	C	Sanded 2 sides.
C (Repaired)-C, Ext. (Underlayment, Ext.).	C (Repaired)[2]	C	C	Sanded 2 sides.
C-C, Ext. (Sheathing, Ext.)	C	C	C	Unsanded grade. No belt sanding permissible.
B-B, Ext. (Concrete Form, Ext.)	B	B	C	Edge-sealed and, unless otherwise specified, mill-oiled. Sanded 2 sides.

[1] See also paragraphs 4.4 and 4.6.

[2] Face may contain knotholes, worm or borer holes, and other open defects not larger than 1/4 by 1/2 in., sound and tight knots up to 1 1/2 in. in greatest dimension, splits up to 1/16 in. wide, ruptured and torn grain, sander skips up to 5 percent of panel area, pitch pockets if solid and tight, plugs, patches, and shims.

Table 3. Overlaid Plywood — Minimum Quality of Veneers

Grade	Face[1]	Back[1]	Inner Plies
A-A, Ext.—High Density Overlay	A	A	B
B-B, Ext.—High Density Overlay	B	B	B
B-B, Ext.—High Density Concrete Form Overlay	B	B	B
B-B, Ext.—Medium Density Overlay	B	B	B
B-B, Ext.—Medium Density Concrete Form Overlay	B	B	B

[1] For overlaid plywood the grade designation for face or back refers to the veneer directly underlying the surface. All overlaid plywood is surfaced on 2 sides unless otherwise specified. When only 1 side is surfaced, the exposed back may be **C** or better.

5. SAMPLING AND TESTING

5.1 **Sampling.**—Ten test panels shall be taken at random from any shipment. Test panels shall be selected to represent as many variations in grades and thicknesses as possible. Test panels shall also be selected from locations distributed as widely as is practicable throughout the shipment. From each Exterior panel selected, three test pieces shall be cut at random and from each test piece ten test specimens shall be cut. From each Interior panel selected, a 6 in. x 6 in. test piece shall be cut from each end approximately at mid-width of the panel, and from each edge approximately at mid-length of the panel, while a fifth piece shall be cut from somewhere near the middle or center of the panel. Overlaid plywood shipments shall be sampled in the same manner as Exterior plywood.

5.2 **Test for Interior Type.**—The test pieces shall be submerged in water at room temperature for a period of 4 hours, and then dried at a temperature not to exceed 100° F for a period of 20 hours. This cycle shall be repeated until all samples have failed, or have completed 15 cycles.

5.3 **Test for Exterior Type.**

5.3.1 **Cold Soaking Test.**—Five shear specimens shall be cut from each test piece as shown in figure 1.

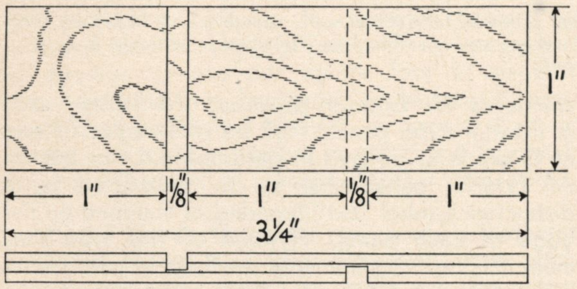


Figure 1.—Shear Specimen.

If the number of plies exceeds 3, the cuts shall be made so as to test any two of the joints, but the additional plies need not be stripped except as demanded by the limitations of the width of the retaining jaws on the testing device. When desired, special jaws may be constructed to accommodate the thicker plywood. If the number of plies exceeds 3, the choice of joints to be tested shall be left to the discretion of the inspector, but at least one-half of the tests shall include the innermost joints. The specimens shall be submerged in water at room temperature for a period of 48 hours and dried for 8 hours at a temperature of 145° F (±5° F), and then followed by two cycles of soaking for 16 hours and drying for 8 hours under the conditions described above. The shear specimens shall be soaked again for a period of 16 hours and tested while wet in a shear-testing device (as illustrated in figure 2) by placing them in the jaws of the device, to which a load shall be applied at the rate of 600 to 1,000 pounds a minute until failure. The percentage of wood failure of the specimens shall be estimated.

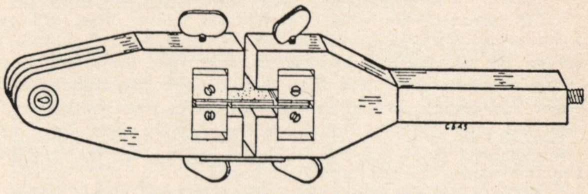


Figure 2.—Jaws for Shear Test

Overlaid plywood shall be evaluated in an identical manner, but in addition to estimating wood failure at the plywood glue lines tested, specimens shall be examined for separation of the resin-impregnated face from the plywood.

5.3.2 **Boiling Test.**—Shear specimens shall be taken as described in paragraph 5.3.1, boiled in water for 4 hours, and then dried for 20 hours at a temperature of 145° F (±5° F). The shear

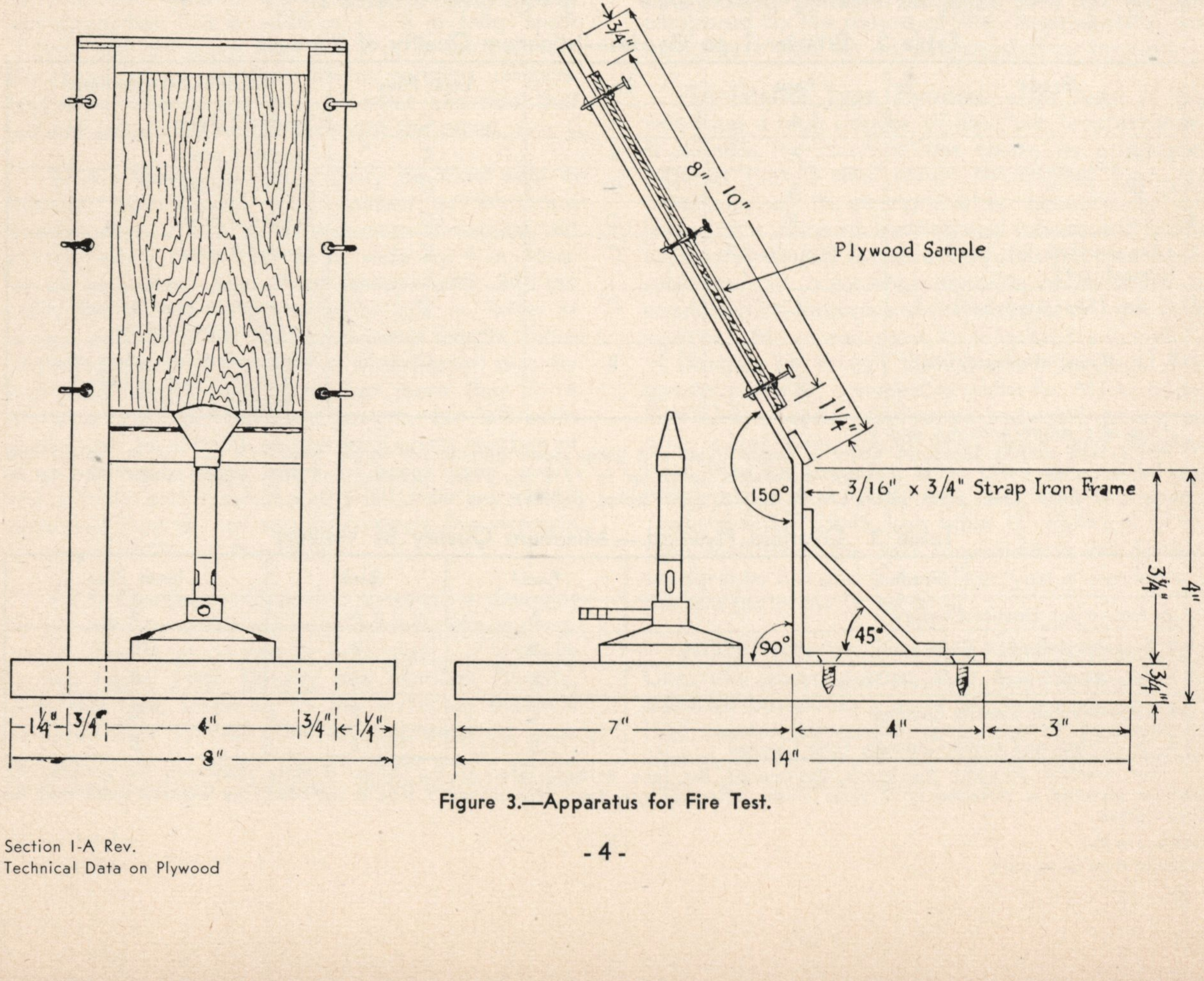


Figure 3.—Apparatus for Fire Test.

specimens shall be boiled again for a period of 4 hours and tested while wet, as described in paragraph 5.3.1. The percentage of wood failure of the specimens shall be estimated.

Overlaid plywood shall be subjected to the above cycles and evaluated as described in paragraph 5.3.1.

5.3.3 **Fire Test.**—A 5 1/2 in. x 8 in. piece shall be taken from each of five selected test panels and shall be placed on the stand as illustrated in figure 3, and subjected to a 800° to 900° C flame from a Bunsen-type burner for a period of 10 minutes or, in the case of a thin specimen, until a brown char area appears on the back side. The burner shall be equipped with a wing top to envelop the entire width of the specimen in flame. The top of the burner shall be 1 in. from the specimen face and the flame 1 1/2 in. high.

The flame shall impinge on the face of the specimen 2 in. from the bottom end. After the test the sample shall be removed from the stand and the glue lines examined for delamination by separating the charred plies with a sharp chisel-like instrument. Any delamination due to combustion shall be considered as failure, except when occurring at a localized defect permitted in the grade. When testing overlaid plywood, blisters or bubbles in the surface caused by combustion shall not be considered delamination.

5.4 **Interpretation of Tests.**

5.4.1 **Interior Type.**—Total visible delamination of 1/4 in. or more in depth, and over 2 in. in

Table 4. Standard Stock Douglas Fir Plywood Sizes—Interior Type

Grade	Width (in.)[1]	Length (in.)[1]	Thickness (in.)[2, 3, 4]				
A-A, Int.	36	72	1/4	---	---	---	3/4
		96	1/4	3/8	1/2	---	3/4
A-A, Int.	48	72	1/4	3/8	1/2	5/8	3/4
		84	1/4	3/8	1/2	5/8	3/4
		96	1/4	3/8	1/2	5/8	3/4
		108	1/4	3/8	---	---	3/4
		120	1/4	3/8	1/2	5/8	3/4
		144	1/4	---	---	---	3/4
A-B, Int.	36	96	1/4	3/8	1/2	5/8	3/4
A-B, Int.	48	72	1/4	3/8	1/2	5/8	3/4
		84	1/4	3/8	1/2	5/8	3/4
		96	1/4	3/8	1/2	5/8	3/4
		108	1/4	---	---	---	3/4
		120	1/4	3/8	1/2	5/8	3/4
		144	1/4	---	---	---	3/4
A-D, Int.	30	60	---	---	---	---	3/4
		72	---	---	---	---	3/4
		84	1/4	---	---	---	3/4
		96	1/4	---	---	---	3/4
		120	---	---	---	---	3/4
A-D, Int.	36	60	---	---	---	---	3/4
		72	1/4	3/8	1/2	5/8	3/4
		84	1/4	3/8	1/2	5/8	3/4
		96	1/4	3/8	1/2	5/8	3/4
		120	1/4	---	---	---	3/4
A-D, Int.	48	60	1/4	3/8	1/2	5/8	3/4
		72	1/4	3/8	1/2	5/8	3/4
		84	1/4	3/8	1/2	5/8	3/4
		96	1/4	3/8	1/2	5/8	3/4
		108	1/4	3/8	1/2	5/8	3/4
		120	1/4	3/8	1/2	5/8	3/4
		144	1/4	3/8	1/2	5/8	3/4
B-B (Concrete Form, Int.)	48	96	---	---	---	5/8	3/4
B-D, Int.	48	84	1/4	3/8	1/2	5/8	3/4
		96	1/4	3/8	1/2	5/8	3/4
C (Repaired)-D (Underlayment, Int.)	48	96	1/4	3/8	1/2	5/8	3/4
C-D (Sheathing, Int.), unsanded	48	96	5/16	3/8	1/2	5/8	3/4
		120	5/16	---	---	5/8	---

[1] A tolerance of 1/32 (0.0312) in. over or under the specified width and/or length shall be allowed, but all panels, including overlays, shall be square within 1/8 (0.125) in. All panels shall be sawn so that a straight line drawn from one corner to the adjacent corner shall fall within 1/16 in. of panel edge.

[2] A tolerance of 1/64 (0.0156) in. over or under the specified thickness shall be allowed on sanded panels, and a tolerance of 1/32 (0.0312) in. on unsanded and overlaid panels.

[3] Minimum number of plies required for standard construction:
- 3 plies for 1/4-, 5/16-, and 3/8-in.
- 5 plies for 1/2-, 5/8-, and 3/4-in.
- 7 plies for 7/8- to 1 3/16-in.

[4] Sanded 2 sides, except C-D (Sheathing).

Note.—Any size panel conforming in all other respects to the various requirements of this standard shall be considered as conforming to this standard.

length along the edge of a 6-in. x 6-in. test piece shall be considered as failure. When delamination occurs at a localized defect permitted within the grade, that test piece shall be discarded. The average number of cycles which the test pieces shall withstand is ten or more, and at least 85% of the specimens shall withstand three cycles. If the test pieces fail to meet these requirements, an additional ten panels shall be selected and tested as described in paragraphs 5.1 and 5.2. Then the test pieces from both groups of ten, considered together, shall meet the above test requirements.

5.4.2 **Exterior Type.** — Specimens cut through localized defects permitted in the grade shall be discarded. A test piece shall be rated by the combined results of both the cold soaking test and the boiling test—generally ten specimens in all. If the average wood failure of the ten specimens is below 60 percent, or if more than one of the specimens is below 30 percent, the test piece fails. If more than one test piece fails, that panel fails. If one or none of the ten panels fails, the shipment is accepted; if more than two fail, the shipment is rejected. If two fail, another series of ten panels is tested. If one or none of the panels fails in this series, the shipment is accepted; otherwise it is rejected. If the average wood failure of the first ten panels is less than 80 percent, a second series of ten is tested regardless of the number of failures. If the average wood failure of the twenty panels combined is less than 80 percent, the shipment is rejected.

The same interpretation shall apply to overlaid plywood. In addition, separation of the resin-impregnated face from the plywood shall be considered failure.

5.4.3 If more than one panel fails the fire test, the shipment may be rejected; if one panel fails, a second series of five shall be tested, all of which must pass.

6. STANDARD STOCK SIZES[1]

6.1 Douglas fir plywood is commonly made in the sizes listed in tables 4, 5, and 6, but other sizes, including 4-, 14-, and 16-foot lengths, may also be available from mills on order.

---

[1] Sizes most commonly available from distributors.

7. INSPECTION

7.1 All plywood guaranteed to conform to the Commercial Standard grading rules is sold subject to inspection in the white only, except concrete-form material, which may have a priming of oil or other preparation before shipment. All complaints regarding the quality of any shipment must be made within 15 days from receipt thereof.

7.2 If the grade of any plywood shipment is in dispute and a reinspection is demanded, the cost of such reinspection shall be borne by the seller and the shipment settled for on the basis of the

Table 5. Standard Stock Douglas Fir Plywood Sizes—Exterior Type

Grade	Width (in.)[1]	Length (in.)[1]	Thickness (in.)[2, 3, 4]						
A-A, Ext.	48	60	1/4	3/8	1/2	5/8	3/4	---	---
		84	1/4	3/8	1/2	5/8	3/4	---	---
		96	1/4	3/8	1/2	5/8	3/4	7/8	1
		108	1/4	3/8	1/2	5/8	3/4	---	---
		120	1/4	3/8	1/2	5/8	3/4	---	---
		144	1/4	3/8	1/2	5/8	3/4	---	---
A-B, Ext.	48	84	1/4	3/8	---	---	3/4	---	---
		96	1/4	3/8	1/2	5/8	3/4	---	1
		120	1/4	3/8	1/2	5/8	3/4	---	---
		144	1/4	3/8	1/2	---	3/4	---	---
A-C, Ext.	36	96	1/4	3/8	1/2	5/8	3/4	---	---
A-C, Ext.	48	72	1/4	3/8	1/2	5/8	3/4	---	---
		84	1/4	3/8	1/2	5/8	3/4	---	---
		96	1/4	3/8	1/2	5/8	3/4	---	1
		108	1/4	3/8	1/2	5/8	3/4	---	---
		120	1/4	3/8	1/2	5/8	3/4	---	---
		144	1/4	3/8	1/2	5/8	3/4	---	---
B-B (Concrete Form, Ext.)	48	96	---	---	---	5/8	3/4	---	---
C (Repaired)-C (Underlayment, Ext.)	48	96	1/4	3/8	1/2	5/8	3/4	---	---
C-C (Sheathing, Ext.) Unsanded	48	96	5/16	3/8	1/2	5/8	3/4	---	---

[1] A tolerance of 1/32 (0.0312) in. over or under the specified width and/or length shall be allowed, but all panels, including overlays, shall be square within 1/8 (0.125) in. All panels shall be sawn so that a straight line drawn from one corner to the adjacent corner shall fall within 1/16 in. of panel edge.

[2] A tolerance of 1/64 (0.0156) in. over or under the specified thickness shall be allowed on sanded panels, and a tolerance of 1/32 (0.0312) in. on unsanded and overlaid panels.

[3] Minimum number of plies required for standard construction:
3 plies for 1/4-, 5/16-, and 3/8-in.
5 plies for 1/2-, 5/8-, and 3/4-in.
7 plies for 7/8- to 1 3/16-in.

[4] Sanded 2 sides, except C-C (Sheathing).

Note.—Any size panel conforming in all other respects to the various requirements of this standard shall be considered as conforming to this standard.

Table 6. Standard Stock Douglas Fir Plywood Sizes—Overlaid Plywood

Grade	Width (in.)[1]	Length (in.)[1]	Thickness (in.)[2]
A-A—High Density, Ext.	36	96	5/16 (3-ply)[3]
A-A—High Density, Ext.	48	96	3/8 (3-ply) 1/2 (5-ply) 9/16 (5-ply) 5/8 (5-ply) 3/4 (5-ply) 7/8 (7-ply) 1 (7-ply) 1 1/8 (7-ply)
B-B—High Density, Ext.	36 48	96	Same as for grade A-A, above.
B-B—High Density, Ext. (Concrete Form)	48	96	1/2 (5-ply) 9/16 (5-ply) 5/8 (5-ply) 3/4 (5-ply)
B-B—Medium Density, Ext.	36 48	96	Same as for grade A-A, above.
B-B—Medium Density, Ext. (Concrete Form)	48	96	1/2 (5-ply) 9/16 (5-ply) 5/8 (5-ply) 3/4 (5-ply)

[1] A tolerance of 1/32 (0.0312) in. over or under the specified width and/or length shall be allowed, but all panels, including overlays, shall be square within 1/8 (0.125) in. All panels shall be sawn so that a straight line drawn from one corner to the adjacent corner shall fall within 1/16 in. of panel edge.

[2] A tolerance of 1/64 (0.0156) in. over or under the specified thickness shall be allowed on sanded panels, and a tolerance of 1/32 (0.0312) in. on unsanded and overlaid panels.

[3] Number of plies refers to veneers. Resin-impregnated surfaces are not included.

Note.—Any size panel conforming in all other respects to the various requirements of this standard shall be considered as conforming to this standard.

reinspection report if the shipment is more than 5 percent below grade, or if it contains more than 1 percent of mismanufactured panels containing defects such as short core, lapped core, blisters, delamination, etc., which render the panel unfit for normal use. The buyer need not accept such defective panels shipped as any standard grade listed in this Commercial Standard.

7.3 If reinspection establishes the shipment to be 5 percent or less below grade, and to contain 1 percent or less of mismanufactured panels, the buyer pays the cost of reinspection and pays for the shipment as invoiced.

8. GRADE MARKING AND CERTIFICATION

8.1 In order to assure the purchaser that he is getting Douglas fir plywood of the grade specified, producers may, individually or in concert with their trade association or inspection bureau, issue certificates with each shipment; or grade-mark each panel as conforming to the standard.

8.2 The following sets forth the grade marking and certification symbols adopted by the Douglas Fir Plywood Association to preserve the high standards of quality herein recorded. The grade-mark symbols on the plywood are to insure that the **ultimate consumer** receives the kind of plywood specified.

8.3 To identify the various grades within the Interior type Douglas fir plywood, these grademarks are stamped or branded on all standard size panels:

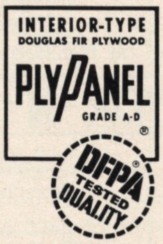
(a) Grade A-A, Int. panels are stamped on the edge:

® **INTERIOR · A-A · DFPA**

(b) Grade A-B, Int. panels are stamped on the edge:

® **INTERIOR · A-B · DFPA**

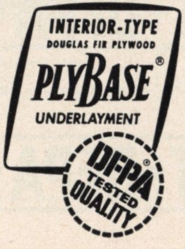
(c) "Plypanel," grade A-D, Int. panels are stamped on the back:



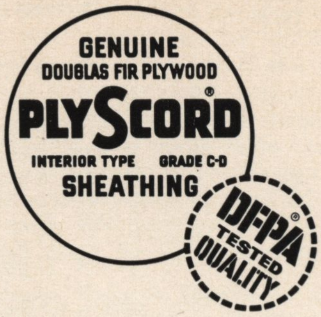
(d) Grade B-D, Int. panels are stamped, usually on the edge:

® **INTERIOR · B-D · DFPA**

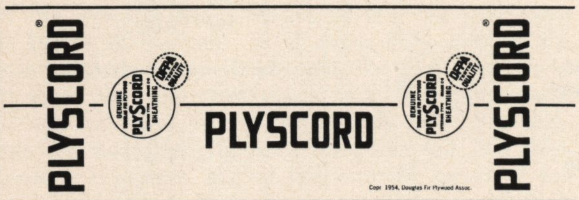
(e) "Plybase" Underlayment, grade C (rep'd)-D, Int. panels are stamped on the back:



(f) "Plyscord," grade C-D, Int. panels are stamped either on the face or back:

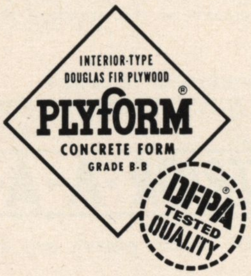


Many Association mills also use across the entire face of a 4-ft. by 8-ft. "Plyscord" sheathing panel at each end the following:



(Lines in the above label represent scorings for nailing to framing members at 16-, 24-, 32-, or 48-in. centers.)

(g) "Plyform," grade B-B, Int. panels are stamped on one face:



(h) Natural Finish One Side, grade N-D, Int. panels are stamped on the edge:

® *INTERIOR* · **N-D** · DFPA

(i) Natural Finish Two Sides, grade N-N, Int. panels are stamped on the edge:

® *INTERIOR* · **N-N** · DFPA

8.4 To identify the Exterior type of Douglas fir plywood, the symbol "EXT-DFPA" is branded or stamped on the edge of each standard size panel. The various grades within the Exterior type are additionally identified by the following grade marks branded or stamped on the edge of each standard size panel.

(a) Grade A-A, Ext. panels are stamped on the edge:

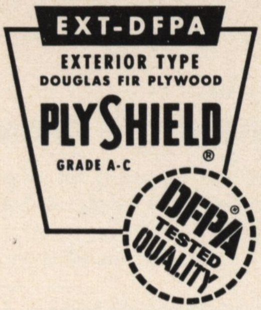
® EXT-DFPA · A-A

(b) Grade A-B, Ext. panels are stamped on the edge:

® EXT-DFPA · A-B

(c) "Plyshield," grade A-C, Ext. panels are stamped on the edge and may also be stamped on the back:

® EXT-DFPA · PLYSHIELD · A-C



OPTIONAL
BACK STAMP

(d) Utility, grade B-C, Ext. panels are stamped on the edge:

® EXT-DFPA · UTILITY · B-C

(e) Grade C (rep'd)-C, Underlayment, Ext. panels are stamped on the edge:

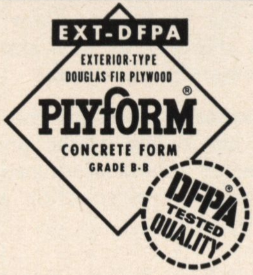
® EXT-DFPA · UNDERLAYMENT

(f) Sheathing, grade C-C, Ext. panels are stamped on the edge:

® EXT-DFPA · SHEATHING · C-C

(g) Concrete Form, grade B-B, Ext. panels are stamped on the edge and may also be stamped on the back:

® EXT-DFPA · PLYFORM · B-B



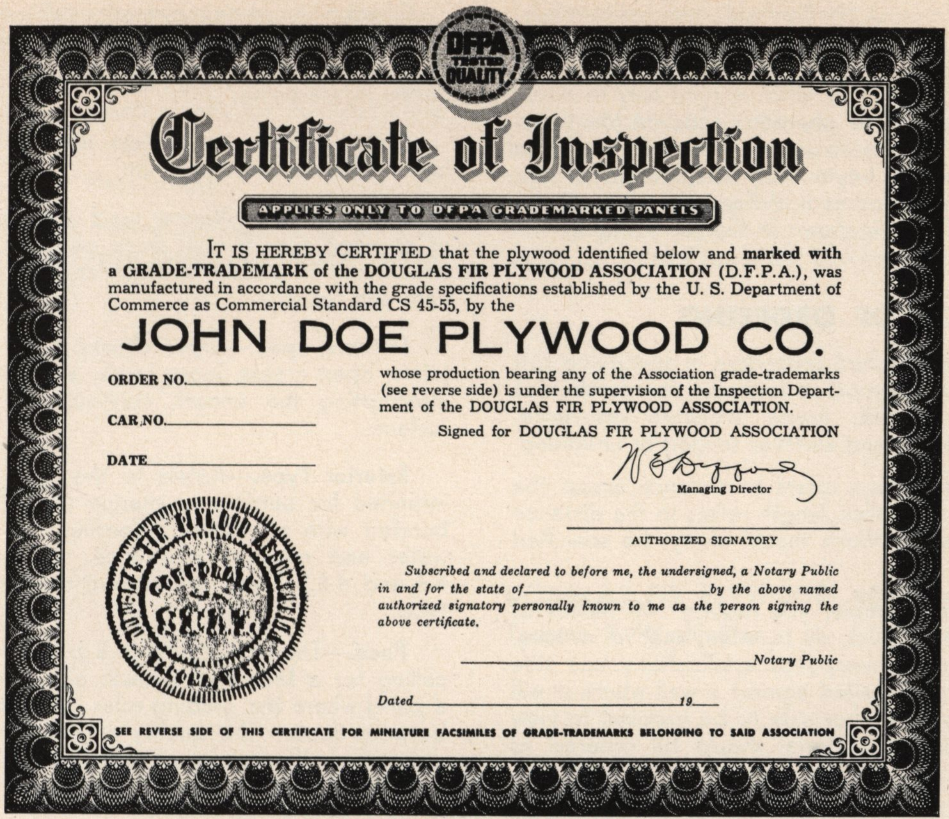
OPTIONAL
BACK STAMP

(h) Each standard size overlaid panel also carries the symbol "EXT-DFPA" branded on the edge to indicate it is of Exterior type.

8.5 **Shop-cutting panel—for remanufacture only.**—Panels stamped as shown below have been rejected as not conforming to grade requirements of standard grades in this Commercial Standard. However they may be especially suitable, through appropriate cutting, for various industrial or other uses. They are analogous to certain shop grades of lumber, also intended for cut up.

**SHOP - CUTTING PANEL
FOR REMANUFACTURE ONLY
DFPA®**

Delaminated or blistered panels are not considered as coming within the category covered by this stamp.


DFPA
TESTED
QUALITY

Certificate of Inspection

APPLIES ONLY TO DFPA GRADEMARKED PANELS

IT IS HEREBY CERTIFIED that the plywood identified below and **marked with a GRADE-TRADEMARK of the DOUGLAS FIR PLYWOOD ASSOCIATION** (D.F.P.A.), was manufactured in accordance with the grade specifications established by the U. S. Department of Commerce as Commercial Standard CS 45-55, by the

JOHN DOE PLYWOOD CO.

ORDER NO.______________

CAR NO.______________

DATE______________

whose production bearing any of the Association grade-trademarks (see reverse side) is under the supervision of the Inspection Department of the DOUGLAS FIR PLYWOOD ASSOCIATION.

Signed for DOUGLAS FIR PLYWOOD ASSOCIATION

Managing Director

____

AUTHORIZED SIGNATORY

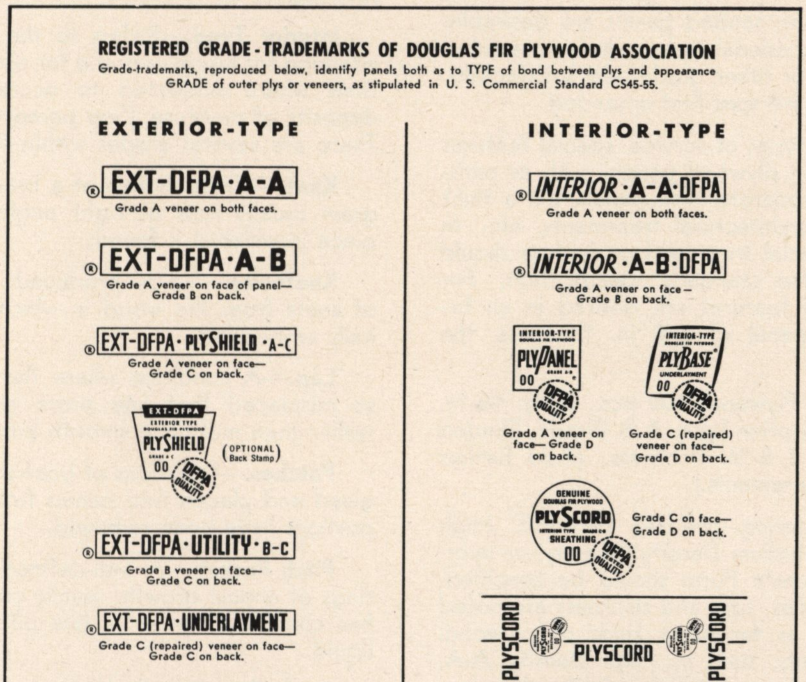
Subscribed and declared to before me, the undersigned, a Notary Public in and for the state of____________by the above named authorized signatory personally known to me as the person signing the above certificate.

____________________*Notary Public*

*Dated*____________________19____

SEE REVERSE SIDE OF THIS CERTIFICATE FOR MINIATURE FACSIMILES OF GRADE-TRADEMARKS BELONGING TO SAID ASSOCIATION

Copyright 1955 Douglas Fir Plywood Association—Printed in U. S. A.



REGISTERED GRADE-TRADEMARKS OF DOUGLAS FIR PLYWOOD ASSOCIATION

Grade-trademarks, reproduced below, identify panels both as to TYPE of bond between plys and appearance GRADE of outer plys or veneers, as stipulated in U. S. Commercial Standard CS45-55.

EXTERIOR-TYPE

® EXT-DFPA·A-A
Grade A veneer on both faces.

® EXT-DFPA·A-B
Grade A veneer on face of panel—Grade B on back.

® EXT-DFPA·PLYSHIELD·A-C
Grade A veneer on face—Grade C on back.

(OPTIONAL Back Stamp)

® EXT-DFPA·UTILITY·B-C
Grade B veneer on face—Grade C on back.

® EXT-DFPA·UNDERLAYMENT
Grade C (repaired) veneer on face—Grade C on back.

® EXT-DFPA·SHEATHING·C-C
Grade C veneer on both faces of panel.

® EXT-DFPA·PLYFORM·B-B
Grade B veneer on both faces of panel.

(OPTIONAL Back Stamp)

INTERIOR-TYPE

® *INTERIOR*·A-A·DFPA
Grade A veneer on both faces.

® *INTERIOR*·A-B·DFPA
Grade A veneer on face—Grade B on back.

Grade A veneer on face—Grade D on back.

Grade C (repaired) veneer on face—Grade D on back.

Grade C on face—Grade D on back.

(Full panel width marking on face)

® *INTERIOR*·N-D·DFPA
Special "natural finish" veneer on face with D back.

® *INTERIOR*·N-N·DFPA
Special "natural finish" veneer on each face.

Grade B on both faces of panel—Grade C for inner plys.

Douglas Fir Plywood Association,
Tacoma 2, Washington

8.6 The Douglas Fir Plywood Association maintains a quality control and an inspection service for the careful grading of its members' products. The certificate of inspection which applies **only** to Association grade-marked plywood, and which is used with carload lots, is to insure that the distributor or first unloading buyer receives plywood of the type and grade specified. A facsimile of the Association certificate is shown in figures 4 and 5.

9. METHOD OF ORDERING

9.1 The established procedure in specifying size and grade of plywood is to name the number of plies, width, length, grade, moisture resistance, finished thickness, and whether sanded or unsanded.

9.2 Width always refers to distance across the grain of the face plies; length refers to the distance along the grain. Width should always be specified first.

9.3 If, for example, you require 100 pieces of plywood 1/4 in. thick, 48 in. wide, and 96 in. long, for interior or semi-exposed applications, one side of which is to be nailed against a wall where it will not show, but the other side to be exposed to view and painted, this material should be ordered as follows:

> Douglas Fir Plywood: 100 pcs., 3-ply, 48 in. by 96 in., Interior Type, A-D Grade, Sanded 2 Sides to 1/4 in. thickness.

9.4 For most uses, sanded panels are desirable, but there are occasional uses where unsanded panels, of an **A-D** or other grade, are satisfactory. Such panels should be specified unsanded.

9.5 For special types of service, special features may be desirable in plywood panels, such as omission of oiling for concrete-form panels, extra thick faces for certain architectural treatments, etc. In such cases, the special treatment or feature should be stated after the standard specification. For example, if special features are desired in an Exterior type **A-A** panel of 3/8 in. thickness, the order should read:

> Douglas Fir Plywood: 100 pcs., 3-ply, 48 in. by 96 in., Exterior Type, A-A Grade, Sanded 2 Sides to 3/8 in. thickness. (Add further special requirements.)

9.6 When ordering overlaid plywood, High Density Overlay, Medium Density Overlay, or overlaid plywood Concrete Form should be specified. The number of pieces, size, and thickness are noted in the same way as for other kinds of plywood. Special requirements, such as High Density **A-A**, surfaced 1 side only, or special weights of surfacing material, should be stated after the standard specification.

10. NOMENCLATURE AND DEFINITIONS

Back.—The side reverse to the face of the panel.

Borer Holes.—Voids made by wood-boring insects or worms.

Centers.—See "Cores".

Check.—A partial separation of veneer fibers, usually small and shallow, running parallel to the grain of the wood, caused chiefly by strains produced in seasoning.

Cores.—Cores or centers are the innermost layer in plywood construction.

Crossbanding.—Veneer used in the construction of plywood with five or more plies. In 5-ply construction it is placed at right angles between the core and faces.

Defects, open.—Open checks, open splits, open joints, open cracks, loose knots, and other defects interrupting the smooth continuity of the panel surface.

Exterior Type.—Refers to the type of plywood intended for outdoor or marine uses. This type is bonded with adhesives, affording the ultimate in water and moisture resistance. (See paragraphs 4.4 and 4.6.) There are several grades within this type.

Face.—The better side of a panel in any grade calling for a face and a back; also, either side of a panel where the grading rules draw no distinction between faces. The quality of the face and back determines the grade of panel within either the Exterior or Interior type.

Heartwood.—The darker-colored wood occurring in the inner portion of the tree, sometimes referred to as "heart".

Interior Type.—Refers to the type of plywood intended for inside uses and for construction applications where subjected to occasional wetting or deposits of moisture. (See paragraphs 4.4 and 4.5.) There are several grades within this type.

Knot.—Cross section of a branch or limb whose grain usually runs at right angles to that of the piece in which it is found.

Knotholes.—Voids produced by the dropping of knots from the wood in which they were originally embedded.

Lap.—A condition where the veneers used are so misplaced that one piece overlaps the other rather than making a smooth butt joint.

Patches.—Insertions of boat-shaped sound wood glued and placed into panels from which defective portions have been removed.

Pitch Pocket.—A well-defined opening between rings of annual growth, usually containing, or which has contained, more or less pitch, either solid or liquid.

Pitch Streak.—A well-defined accumulation of pitch in a more or less regular streak.

Plugs.—Sound wood, usually circular, for replacing defective portions which have been removed. Plugs usually are held in veneer by friction only until veneers are bonded into plywood. Synthetic plugs are of fiber and resin aggregate; they are used to fill openings and provide a smooth, durable surface.

Sapwood.—The lighter-colored wood occurring in the outer portion of the tree, sometimes referred to as "sap".

Shim.—A long, narrow repair not more than 3/16 in. wide.

Split.—Complete separation of veneer fibers parallel to grain, caused chiefly by manufacturing process or handling.

Streaks.—See "Pitch streak".

Torn Grain.—A marked leafing or separation on veneer surface between spring and summer wood.

Veneer.—Thin sheets of wood.

Veneer Patches. — Patches inserted in veneer sheet before panel is assembled for pressing.

60/60, 65/65, 93/93, etc.—Such optional symbols may be used by manufacturers of overlaid plywood to indicate the weight of the overlay in pounds per 1,000 sq. ft. on each side of the panel. The weight of the overlay includes resin and carrier sheet (or sheets) together, before pressing.

DOUGLAS FIR PLYWOOD ASSOCIATION
Tacoma 2, Washington

SECTION TWO

Revised November 1, 1948

DESIGNING WITH PLYWOOD

INTRODUCTION:

Working stresses for plywood are based on the strength properties and "basic stresses" for wood, as determined by the U. S. Forest Products Laboratory.

These "basic stresses" for Douglas fir and suggested methods of applying them to plywood design have been outlined by the Forest Products Laboratory. This outline is printed herewith on pages 6 and 7, with the Laboratory's permission.

With this fundamental information, and a knowledge of how plywood functions both lengthwise and crosswise of the panel, the engineer or architect is in a position to design with plywood, as with any other structural material.

STRENGTH PROPERTIES:

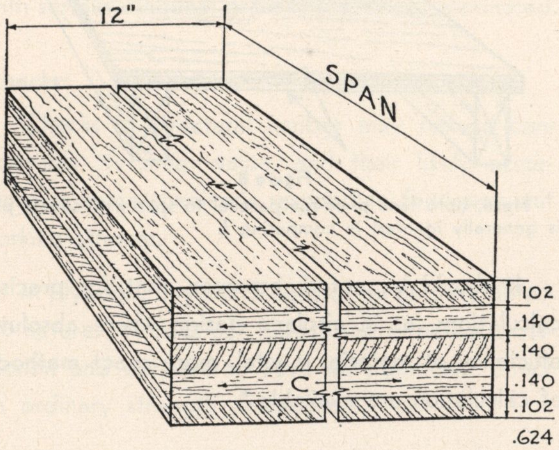
Tension, Compression, and Flexure:

When plywood is subject to a tensile or compressive force lengthwise, only those plies having their grain running lengthwise are considered as carrying the load. The cross plies, of course, are stressed across the grain and so are incapable of contributing any significant amount to the strength of plywood in tension or compression.

Similarly, in bending or flexure, only the plies running parallel to the span are considered in most cases in computing the moment of inertia, I, of the cross-section. When plywood is used so that the grain of the exterior plies (face grain) runs parallel to the span, the error (on the safe side) in neglecting the perpendicular-to-the-span plies is quite small, being less than 1% in a 1/4″ panel, about 4% in a 1″ panel, and almost never exceeding 5%.

When plywood is used in flexure with its face grain perpendicular to the span (i. e., parallel to the support), the face plies will contribute considerably to the stiffness, and also to the strength of 3-ply panels, provided in the latter case, that the face plies consist of single sheets of veneer, or of strips edge-glued to form wide faces. For example, in a 3/8″ panel with three 1/8″ plies, the I ⊥ * for a 12″ width, considering only the center ply is .0020 in.^{4}

Typical Computation for Moment of Inertia, I



$$I\ (\text{for } 12'' \text{ width}) = \frac{12}{12}\left[\left(\frac{5}{8}\right)^3 - (3 \times 0.14)^3 + 0.14^3\right]$$

$$\text{or, } I = .173 \text{ in.}^4$$

If the grain of the face plies runs at right angles to the span then only the cross-bands, "c", would be considered in computing the

$$I = \frac{12}{12}\left[(3 \times 0.14)^3 - 0.14^3\right] = 26 \times 0.14^3 = .071 \text{ in.}^4$$

If, however, we consider the face plies also, and the fact that the modulus of elasticity, E, for wood in flexure across the grain equals about 1/20 of that along the grain, the effective $I\perp = .0020 + (1/20\ I\ \| = .0025) = .0045$ in.^{4}. In 5-ply panels, the increase in I⊥ from adding 1/20 I ∥ is much less significant as will be seen from values in Table, page 10.

Because plies running perpendicular to the span (figs. A and B) contribute only a small amount to the bending strength and stiffness of the panel, it is considered sound and entirely satisfactory practice generally to disregard their effect, except in considering the stiffness of 3-ply panels with face grain ⊥ to span, as noted in the previous paragraph. (See also Forest Products Laboratory reprint, Table I, "Deflection in Bending.")

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*For convenience in referring to the moments of inertia of plywood lengthwise and crosswise, I ∥ will be used to designate the I when the face grain is parallel to the span, and I⊥ will designate I where the face grain is perpendicular to the span.

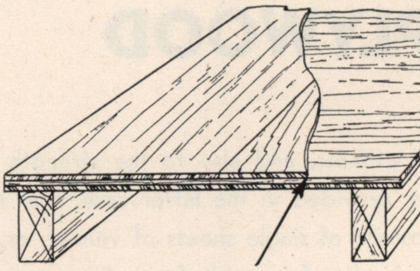

Figure A

Here, only center ply runs parallel to span. Face plies, therefore, are generally ignored in computing I.

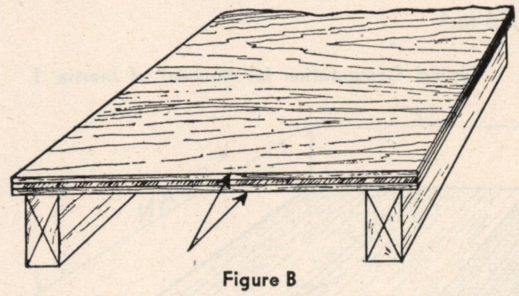

Figure B

Here, both face plies run parallel to span and center ply is generally ignored in computing I.

It should be noted, however, that for precise calculations, as in airplane design where absolute efficiency of material is vital, more exact methods of calculation are available.*

SHEAR:

Shearing stresses in plywood are of two distinctly different types, either one of which may be the critical factor in design. One kind of shear is that ⊥ to the plane of the panel, as horizontal (or vertical) shear in a built-up beam with a plywood web. This shear is computed over the full cross-sectional area of the panel, using an allowable stress, when the direction of stress is parallel or perpendicular to the face grain, of twice the basic horizontal shear stress for clear Douglas fir, i. e., 2×120 psi, with suitable reduction for the plywood grades. If the direction of stress is at 45° to the face grain, four times the basic horizontal stress is allowed. A common example of this type of shear is in plywood used as wall sheathing or as a diaphragm in designing against earthquakes.

The second type of shear is that in the plane of the panel. In plywood, the wood fibers in the ply at right angles to the principal shearing force tend to roll, and a so-called "rolling shear" is induced. Resistance of plywood against this type of shear is about one-third that offered by wood parallel to the grain, but because of beam checking and other factors not present in plywood which reduce the basic stress for lumber considerably, the Forest Products Laboratory suggests a unit shear in "plane of plies" of ¾ths the basic horizontal shear stress of 120, or 90 psi.

The adhesives used in making Douglas fir plywood bond the veneers together in a joint stronger than the wood itself, so that the wood fibers themselves are the limiting factor in rolling shear strength.

Because of the lesser shear strength where "rolling shear" occurs, the critical stress in a panel with a stressed plywood cover will occur between the joist and the innermost plywood face when the face grain is perpendicular to the joist, and between the innermost face and the adjacent ply when the face grain is parallel to the joist. See

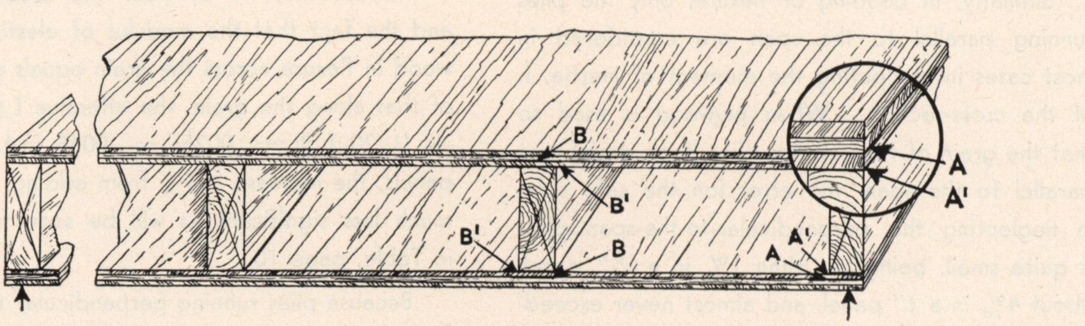

Figure C

Built-up panel with stressed covers (plywood glued to frame). If the plywood has its face plies parallel to the joists, the critical shear ordinarily will be at A or B, in the plane of the plies, rather than at A' or B' where the fibers in the face of the plywood and in the joist are parallel. At A, above the "outside joist," only ¼ the unit shear stress is to be used, whereas at B, where "symmetrical" stress concentration is assumed, ½ the unit shear stress would be used. (See Table I.)

If the plywood runs ⊥ to the joists, these same unit shear stresses would then apply at A' and B', where the panels are glued to the joists and where "rolling shear" would occur.

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*See ANC Bulletin "Design of Wood Aircraft Structures" which may be obtained for $1.00 from the U. S. Government Printing Office, Wash., D. C.

Fig. C. For these cases, the shear stress above an interior joist is recommended as ¾ths the basic shear stress.

The effective shear area in a plywood cover plate is only that area immediately above the joist and of a width equal to the joist width.

In an I or box beam, where the flange joins the web, stress concentrations occur because of the sudden change of dimensions. This applies also to the glued joists at the **outer** joists in built-up panels with stressed plywood covers. Accordingly the Forest Products Laboratory suggests that only ⅜ths the basic shear stress be used at such joints, under the conditions detailed in Table I, p. 7. (See also Fig. C).

Rolling shear stresses when they occur at 45° to the face grain direction, are somewhat higher, as shown in Table I.

ALLOWANCE FOR GRADE

Working stresses for the various structural grades of solid lumber are predicated on allowable knots, slope of grain, pitch pockets and similar characteristics of each grade which reduce the effective cross-section of the piece.

In plywood, the same principles apply, but due to the nature of the plywood, such localized defects probably have a relatively small influence on the strength of the panel as a whole.

Comments on the known effect of certain of these characteristics follow:

Slope of Grain:

In structural lumber the grading rules limit the "slope of grain," which is simply the angle, measured along the sides, which the grain makes with the outer surfaces of the piece.

The grain of rotary cut Douglas fir veneer is normally parallel to the surface. Sloping grain may occur in a panel in small localized areas, but one or two inches away the grain may be straight again, so that it is difficult to estimate its degree or extent, and there is no way of eliminating these local deviations from straight grain. Fortunately, any effect they may have on strength generally appears to be slight except in small, narrow specimens or where the entire panel is to be bent to a sharp degree of curvature, as in some bent planking. Hundreds of flexure tests on unselected plywood panels secured from numerous mills showed moduli of rupture from 8500 to 15,500 lbs. per square inch, and fiber stresses at P. L. from 6900 to 8500. The average values compare favorably with those for straight grain solid Douglas fir. Moreover, none of the test pieces revealed any weaknesses that could be attributed to slope of grain. Accordingly, it appears that in Douglas fir plywood, slope of grain generally need not be considered and that any effect it may have on strength is reflected in average test values, to which safety factors are applied in determining proper working stresses.

Similarly, diagonal face grain is a possibility, but actually its presence would cause a panel to warp and twist in proportion to the severity of the grain distortion, so that in manufacture, veneer with serious diagonal grain is practically eliminated.

Checks:

Checks in structural lumber may reduce considerably its shear value, and their likely occurrence in some species is responsible for lower unit working stresses.

In plywood, any checking is of an entirely different nature. It does not reduce the effective cross-section and appears to have no significant effect on ordinary strength properties.

Knots and Other Grading Factors:

Certain characteristics such as knots, knotholes, and pitch pockets, reduce the cross-sectional area. Accordingly the basic stresses for "clear" material must be reduced a proper percentage for each of the several plywood grades, to allow for the factors permitted in that grade under the current U. S. Commercial Standard.

The face plies, of course, are of major importance in bending, since the other "parallel" plies contribute little to the strength. For example, using the method illustrated on Page 1, a panel consisting of seven plies, each ⅛″ thick, would have an $I \parallel = (7/8)^3 - (5/8)^3 + (3/8)^3 - (1/8)^3 = 244/512$ (or .477) ins.^{4}; but of this, the two face plies contribute $(7/8)^3 - (5/8)^3 = 218/512$ while the two inner parallel plies provide only $(3/8)^3 - (1/8)^3 = 26/512$, i. e., about 11% of the total moment of inertia. Similarly, in a 5-ply panel with five $1/8''$ veneers, the $I \parallel = (5/8)^3 - (3/8)^3 + (1/8)^3 = 99/512$, with the single inner parallel ply contributing $(1/8)^3 = 1/512$ to the total I, or about one percent. Normal defects in interior plies, therefore, are insignificant for ordinary construction uses where flexure is concerned.

In plywood grades, the number and type of characteristics are limited chiefly by appearance requirements, by manufacturing economy, and by their natural occurrence in Douglas fir veneer. Special structural grades of plywood, designed to develop maximum strength properties, are rarely in demand. Moreover, except for gusset plates with Teco ring connectors, it appears doubtful that such special grades would be justified. It is simpler in most instances to use a slightly thicker panel in order to develop the extra load capacity desired.

Experience shows, and inspection of any lot of plywood will tend to confirm, that even in Sheathing, (see Figure D) the grade of plywood in which the greatest number of knots and other characteristics occur, it is difficult to find one face in any panel with enough defects of all kinds to aggregate 6 inches in a 48″ width. If equivalent characteristics were present in the opposite face and at the same cross-section, there still would be only a $1/8$ reduction in the effective cross-sectional area in bending. Characteristics permitted in crossbands or center plies are of little significance in bending, since, as already noted, such plies contribute but slightly, if at all, to the bending strength. A 1″ knot in a 24″ wide panel causes a greater percentage of reduction than if in a 48″ wide panel, but nevertheless experience indicates that for Plyscord sheathing it is conservative to allow a 25% reduction from the "clear" grade unit stresses, and proportionately lower reductions for the higher grades with their lesser defects.

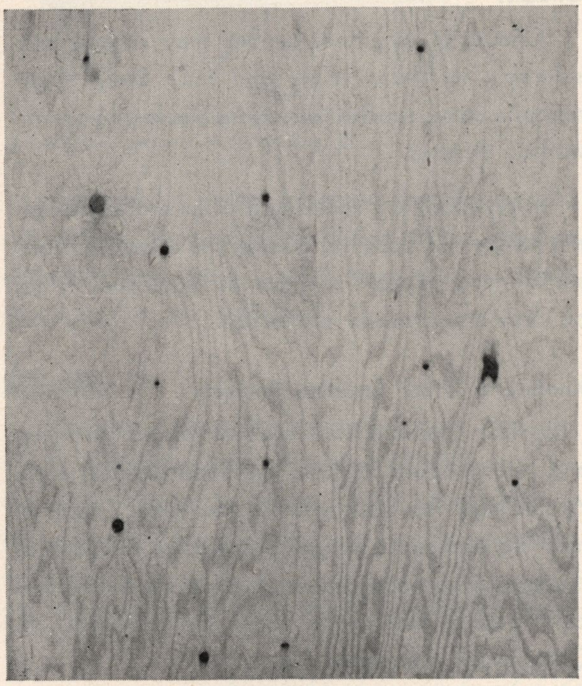


Figure D
Example of portion of Sheathing Panel

Since stiffness is not appreciably affected by knotholes, pitch pockets and the like, no change is made in values of modulus of elasticity for the different grades.

TENSION AND COMPRESSION:

For panels of 5 or more plies, it is recommended that the unit stress in compression parallel be taken for the next lower grade, for reasons outlined below, i. e., for EXT-DFPA•A-A use 1460 lbs. per square inch; for EXT-DFPA•Plyshield, 1375; for Plyscord, no change.

In tension and compression, all plies running parallel to the stress direction assume importance (although where eccentricity of loading is present, the face plies are of relatively greater importance); still, the effect of knots and other characteristics in the inner plies of a panel must be considered. It is believed that even the poorest veneer that might be used for cores and center plies will have a net cross-sectional area equivalent to 75% of that for clear straight-grained veneer. In 3-ply panels, of course, there is no center or inner ply parallel to the faces, whereas in 5-ply, there will be one center ply and two faces, and in 7-ply, there will be two centers and two faces to consider.

Now in Plyscord, it will be noted there has already been a 25% reduction in basic stresses in compression (and also tension) to provide for characteristics permitted in the face, so that the effect of such factors in the **inner** plies will not necessitate any further reduction in stresses.

In the higher grades, however, it is estimated that face characteristics would never require more than 20% reduction for EXT-DFPA•Plyshield, and 12½% for EXT-DFPA•A-A. Theoretically, to get proper unit compression and tension stresses in the higher grades of five or more plies, it would be necessary to consider the thickness, both of the inner 'parallel" plies with their 25% reduction in stress, and of the face plies with their lesser reductions. Unless, however, all of the worst defects in each of the parallel plies occurred in the same cross-section of a panel, the full stress reductions would not be needed.

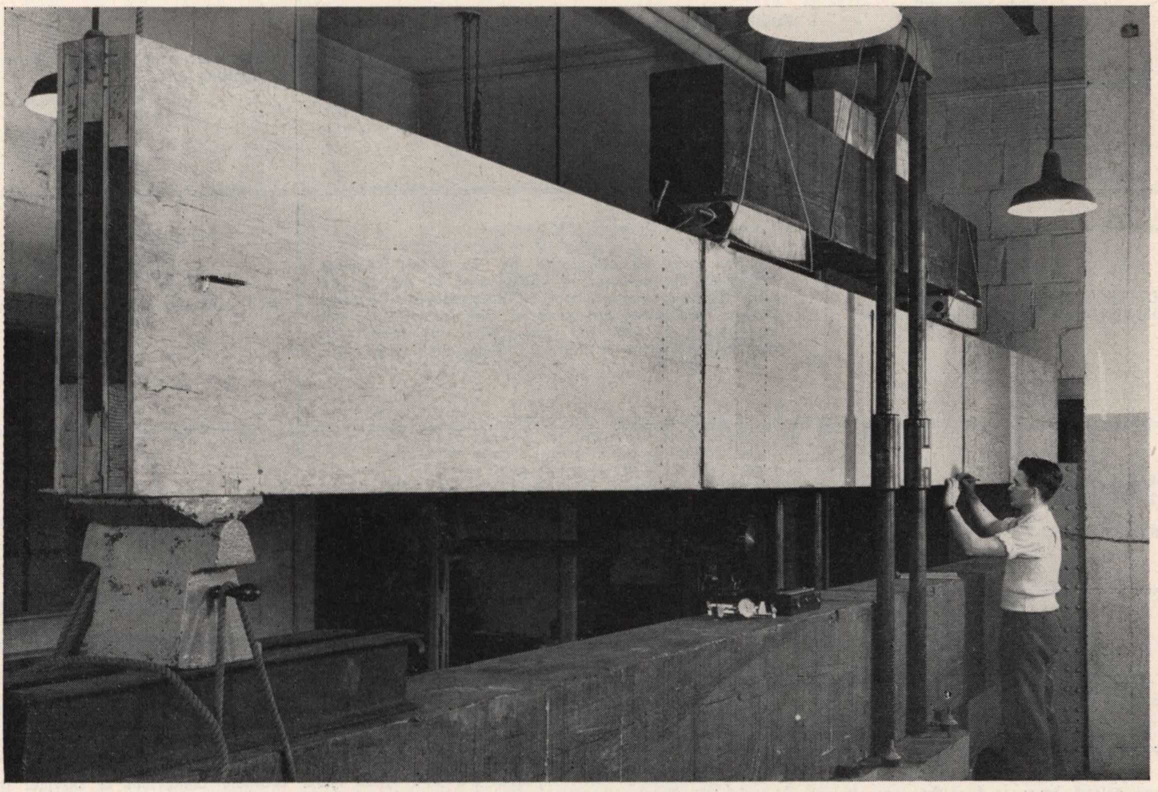
Accordingly, for simplicity it is suggested that, for plywood of five or more plies acting in either tension or compression parallel, the unit stress to be used shall be that given for the next lower grade. For Plyscord, as already noted, there would be no change.

It should be noted that where plywood is subjected to compression in the plane of the panel, but perpendicular to the grain of the face plies, a higher load capacity will be obtained by considering the plies parallel to the applied force, with which a higher working stress is used than with those plies running perpendicular, even though there will always be one more of the latter.

In accordance with standard practice for structural lumber, no reduction for grade is required for working stresses in compression perpendicular.

FOR DRY USES:

It is important to note that the Forest Products Laboratory mimeograph, reprinted on Page 8, allows a 25% increase in certain working stresses for Douglas fir plywood used where its moisture content will not exceed 16%. For wood at 70°F. to attain such a moisture content, the relative humidity in the air must be maintained at least at 78%. Ordinarily, plywood used in floors, ceilings, partitions and walls will not exceed 16% in moisture content, and accordingly, for such uses, the unit working stresses shown on page 11 are recommended:



Thirty inch plywood girder under test at University of Washington Forest Products Laboratory.

Tables 1, 2 and 3 are from Bulletin R-1630, (revised October 1946)

APPROXIMATE METHODS OF CALCULATING THE STRENGTH OF PLYWOOD

By L. J. MARKWARDT, Assistant Director
and A. D. FREAS, Engineer
Forest Products Laboratory, Forest Service
U. S. Department of Agriculture

Table 1—Design method and allowable stresses

PROPERTY	DIRECTION OF STRESS WITH RESPECT TO DIRECTION OF FACE GRAIN	AREA TO BE CONSIDERED	UNIT STRESS TO BE USED
TENSION	Parallel or Perpendicular	Parallel plies[2] only	Unit stress for extreme fiber in bending.
	±45°	Full cross-sectional area	One-sixth unit stress for extreme fiber in bending.
COMPRESSION	Parallel or Perpendicular	Parallel plies[2] only	Unit stress in compression parallel to grain.
	±45°	Full cross-sectional area	One-third unit stress in compression parallel to grain.
BEARING AT RIGHT ANGLES TO PLANE OF PLYWOOD		Loaded area	Unit stress in compression perpendicular to grain.
LOAD IN BENDING	Parallel or Perpendicular	Bending moment $M = KSI/c$ where S = unit stress for extreme fiber in bending; I = moment of inertia computed on basis of parallel plies only; c = distance from neutral axis to outer fiber of outermost ply having its grain in the direction of the span; $K = 1.50$ for three-ply plywood having the grain of the outer plies perpendicular to the span; $K = 0.85$ for all other plywood	Unit stress for extreme fiber in bending.
DEFLECTION IN BENDING	Parallel or Perpendicular	Deflection may be calculated by the usual formulas, taking as the moment of inertia that of the parallel plies plus one-twentieth that of the perpendicular plies. (When face plies are parallel, the calculation may be simplified, with but little error, by taking the moment of inertia as that of the parallel plies only.)	Unit value for modulus of elasticity.
DEFORMATION IN TENSION OR COMPRESSION	Parallel or Perpendicular	Parallel plies[2] only	Unit value for modulus of elasticity.
SHEAR THROUGH THICKNESS	Parallel or Perpendicular	Full cross-sectional area	Double unit stress in horizontal shear.
	±45°	Full cross-sectional area	Four times unit stress in horizontal shear.

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[1] The suggested simplified methods of calculation apply reasonably well with usual plywood types under ordinary conditions of service. It is recognized, however, that they are not entirely valid for all types of plywood and plywood constructions, or for all spans and span-depth ratios. Also the methods given are not applicable to structures so proportioned that the plywood is in the buckling range, in which event the results will be too high.

[2] By "parallel plies" is meant those plies whose grain direction is parallel to the direction of principal stress.

Table I—Design method and allowable stresses —(continued)

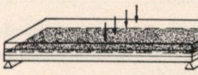
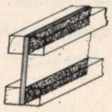
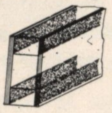
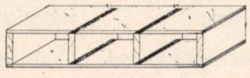
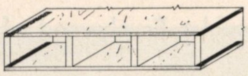
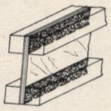
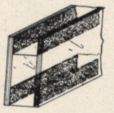
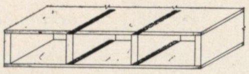
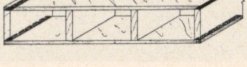
PROPERTY	DIRECTION OF STRESS WITH RESPECT TO DIRECTION OF FACE GRAIN	AREA TO BE CONSIDERED	UNIT STRESS TO BE USED
SHEAR IN PLANE OF PLIES	Parallel or Perpendicular	Full shear area. 1. Plywood beams. Horizontal shear: 	1. Three-fourths unit stress in horizontal shear.
		Area of contact between plywood and flange or framing member. 2. I- or box-beams with plywood webs. Shear between plies of web or between web and flange:  	2. Three-eighths unit stress in horizontal shear.
		3. Panels having plywood covers stressed in compression or tension, or both. Shear between plies or between cover and framing members when depth of member exceeds twice its width and end headers are used or when depth is not more than twice the width and no headers are used. A. Interior framing members: 	3. A. Three-fourths unit stress in horizontal shear.
		B. Framing members at edge of panel: 	B. Three-eighths unit stress in horizontal shear.
	±45°	Area of contact between plywood and flange or framing member. 1. I- or box-beams with plywood webs. Shear between plies of web or between web and flange:  	1. One-half unit stress in horizontal shear.
		2. Panels having plywood covers stressed in compression, or tension, or both. Shear between plies or between cover and framing members when depth of member exceeds twice its width and end headers are used or when depth is not more than twice the width and no headers are used. A. Interior framing members: 	2. A. Unit stress in horizontal shear.
		B. Framing members at edge of panel: 	B. One-half unit stress in horizontal shear.

Table 2—Basic stresses for Douglas fir (Coast Region) free from defects

(Source: USDA Miscel. Pub. 185, "Guide to the Grading of Structural Timbers")

Property	Pounds per square inch
Extreme fiber in bending	2,000
Compression perpendicular to grain	325
Compression parallel to grain	1,466
Maximum horizontal shear	120
Modulus of elasticity	1,600,000

[1] Basic stresses for some other species are given in table 8, USDA Miscel. Pub. 185, "Guide to the Grading of Structural Timbers."

Appropriate unit stress for table 1 may be obtained from table 2 by multiplying the basic values by the appropriate reduction factor, as follows:

(1) For Douglas fir plywood to be used in dry locations (moisture content 16 percent or less), the basic stresses for extreme fiber in **bending, compression perpendicular to grain,** and **compression parallel to grain** may be increased by 25 percent. (No increase for maximum horizontal shear or modulus of elasticity.)

(2) The basic stresses are for clear wood without defects. An appropriate reduction factor is to be used according to the estimated grade of material with respect to defects allowed. When defects present are estimated to reduce the strength one-fourth, multiply the basic stresses by three-fourths, etc.

Example: What unit stress should be used for tension parallel to face grain for plywood of a three-fourths grade to be used in dry locations?

Procedure: 2,000×1.25×0.75=1,875 pounds per square inch.

Table 3—Example of unit stresses for Douglas fir plywood of three-fourths grade to be used in dry locations, for use with table 1

Property	Direction of stress with respect to face grain	Unit stress Pounds per square inch
Tension	Parallel or perpendicular	1,875
Tension	± 45°	310
Compression	Parallel or perpendicular	1,375
Compression	± 45°	460
Shear	Parallel or perpendicular	180
Shear	± 45°	360
Shear in plane of plies	Parallel or perpendicular	[1]34
Bending	Parallel or perpendicular	1,875
Modulus of elasticity	Parallel or perpendicular	1,600,000
Compression perpendicular to grain (bearing)		400

[1] Applies to shear stresses, such as in I- and box-beams, where stress is concentrated at the inner edge of the flange. For other conditions as outlined in table 1, different values would be found as calculated by the appropriate factor.

FACTOR OF SAFETY:

Briefly, the basic unit stresses for Douglas fir lumber (and plywood) have been determined by the U. S. Forest Products Laboratory from thousands of tests on clear, green specimens, with due regard to species characteristics.

For the extreme fiber stress in bending, for example, the average ultimate strength from such tests is reduced by 25% to allow for variability.

Next, to allow for effect of possible long-time loading, a 9/16 factor is used. Then a true factor of safety of 1-2/3 is employed. The result is a value approximately 1/4 of the original ultimate strength.

TABULAR DATA:

Table, page 10, lists the I || and I ⊥, together with section moduli and areas of veneers per foot of width for plywood used both parallel and perpendicular. In computing these tables, veneer thicknesses selected for the various panel thicknesses are for the most part those commonly employed by the manufacturer. When there is a variation in manufacturing practice in certain thicknesses, a conservative construction has been selected.

STIFFNESS:

The values for I give immediately the comparative stiffness of different plywood thicknesses and may be substituted in the standard deflection formulae. (See also F. P. L. Reprint, Table I, "Deflection in Bending," Page 6.)

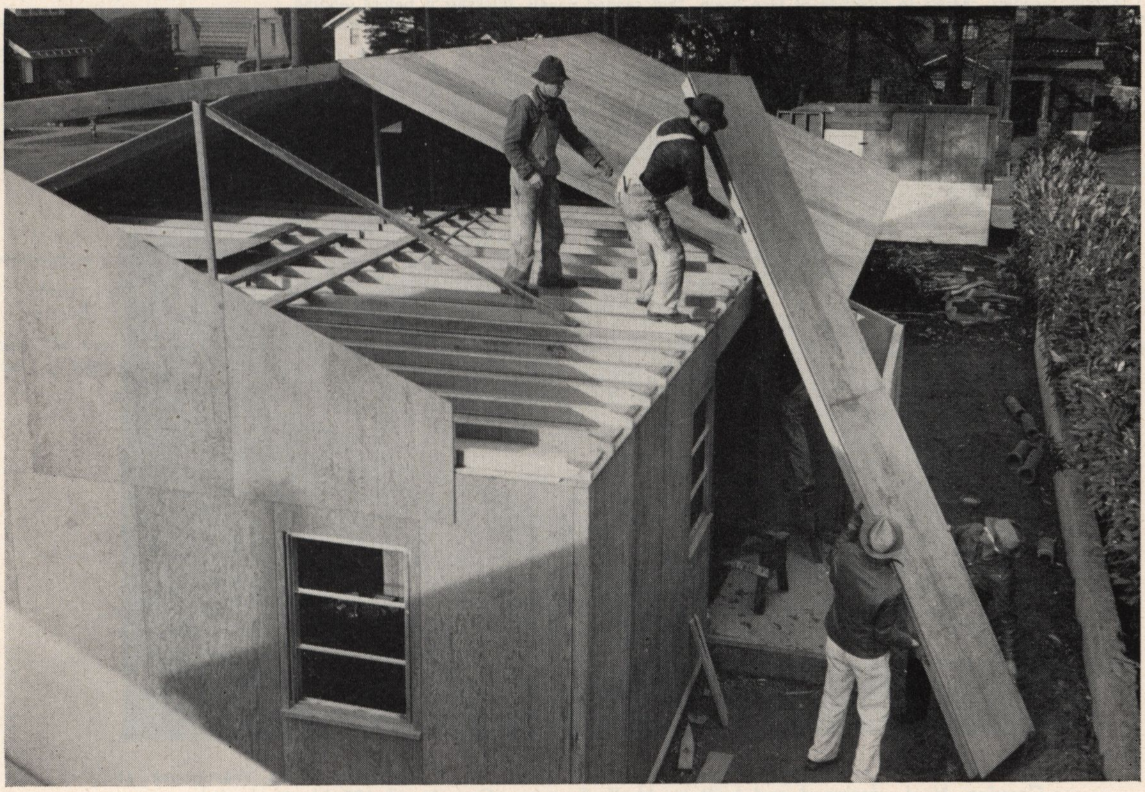
STRENGTH:

Similarly, values for I/c, or section moduli, give relative strengths in bending, and are used with standard formulae, except that the Forest Products Laboratory recommends using the constant K, with bending moment formula, $M=KSI/c$. (See page 6, Table I, "Load in Bending.") It is to be noted that "c" = distance from neutral axis to outer fiber of outermost ply running parallel to span. This is important in panels used perpendicular to span.

It will be noted that $I_{\parallel} + I_{\perp} = I$ of a cross-section of solid wood of equal quality; also that as plywood increases in thickness, its strengths lengthwise and crosswise tend to equalize.

Area of veneer cross-sections will be useful where tension or compression is involved.

Shear in either a horizontal or vertical plane, ⊥ to the face of a panel used flatwise or on edge, is computed across the full cross-sectional area of the panel.



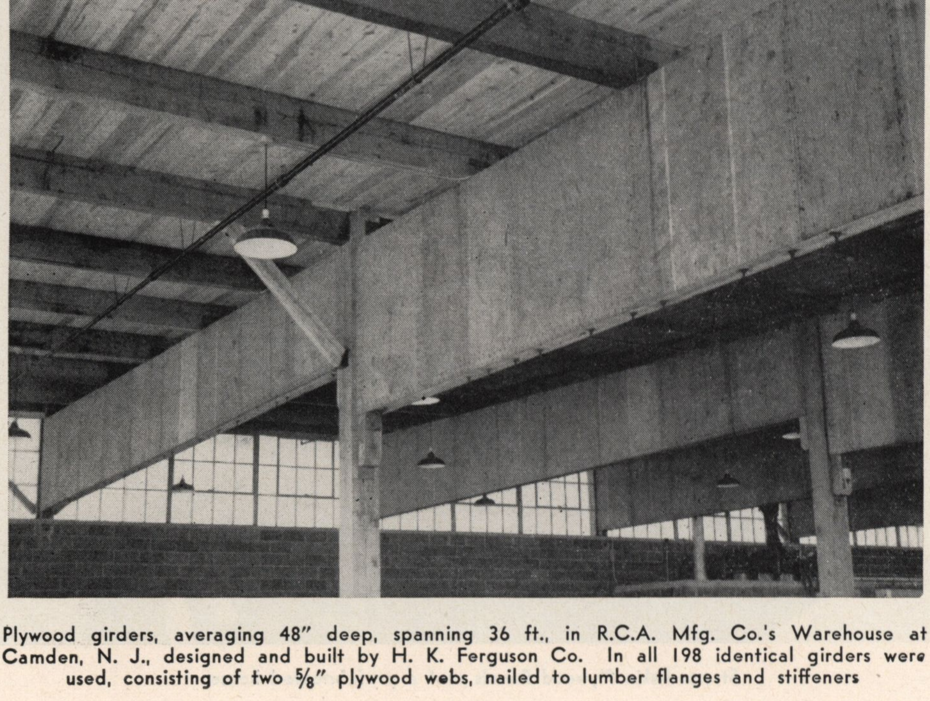
Stressed-skin Plywood Panels roof a prefabricated house.

MOMENTS OF INERTIA, SECTION MODULI and VENEER AREAS *for* SELECTED PLYWOOD CONSTRUCTIONS

(12" WIDTHS)

Plywood Thickness (net)	No. of Plies	VENEER THICKNESS (Nominal) in Inches			PARALLEL[3] PLIES ONLY			PERPENDICULAR[3] PLIES ONLY			Weight, lbs. per 1000 sq. ft. (Approx.) (As shipped from Mill)
		Faces*	Centers	Crossband	Area (Sq. Ins.)	Moment of Inertia I (Inches^4)	Section Modulus S (Inches^3)	Area (Square Inches)	Moment of Inertia I (Inches^4)	Section Modulus S (Inches^3)	
1/8"—R(1)	3	1/24	1/24		1.00	.0019	.030	0.50	.0001	.0034	490
1/8"—S(2)	3	1/16	1/16		0.75	.0017	.027	0.75	.0002	.0077	490
3/16"—R	3	1/16	1/16		1.50	.0064	.068	0.75	.0002	.0077	640
3/16"—S	3	1/12	1/12		1.25	.0060	.064	1.00	.0006	.0139	640
1/4"—R	3	1/12	1/12		2.00	.0150	.120	1.00	.0006	.0139	790
1/4"—S	3	1/9	1/9		1.67	.0143	.114	1.33	.0014	.0247	790
5/16"—R	3	1/10+	1/10+		2.50	.0294	.188	1.25	.0011	.0215	950
5/16"—S	3	1/8	1/8		2.25	.0286	.183	1.50	.0020	.0312	950
3/8"—R	3	1/8	1/8		3.00	.0509	.271	1.50	.0020	.0312	1125
3/8"—S	3	1/8	3/16		2.25	.0461	.246	2.25	.0066	.0704	1125
3/8"—S	5	1/10	1/12	2@1/12	2.50	.0377	.201	2.00	.0150	.120	1125
7/16"—R	3	1/8	3/16		3.00	.0772	.353	2.25	.0066	.0704	1300
7/16"—R	5	1/10	1/12	2@1/12	3.25	.0688	.314	2.00	.0150	.120	1300
7/16"—S	5	1/10	1/10	2@1/10	2.85	.0575	.263	2.40	.0260	.1735	1300
1/2"—R	5	1/10	1/10	2@1/10	3.60	.0990	.396	2.40	.0260	.1735	1525
1/2"—S	5	1/8	1/8	2@1/10	3.60	.0926	.370	2.40	.0324	.1995	1525
9/16"—R	5	1/8	1/8	2@1/10	4.35	.1457	.517	2.40	.0324	.1995	1675
9/16"—S	5	1/8	1/8	2@1/8	3.75	.1273	.452	3.00	.0507	.271	1675
5/8"—R	5	1/8	1/8	2@1/8	4.50	.1934	.619	3.00	.0507	.271	1825
5/8"—S	5	1/8	3/16	2@1/8	4.50	.1670	.534	3.00	.0771	.352	1825
11/16"—R	5	1/8	3/16	2@1/8	5.25	.2478	.720	3.00	.0771	.352	2000
11/16"—S	5	1/8	1/8	2@3/16	3.75	.202	.588	4.50	.123	.492	2000
3/4"—R	5	1/8	1/8	2@3/16	4.50	.299	.798	4.50	.123	.492	2225
3/4"—S	5	1/8	3/16	2@3/16	4.50	.251	.670	4.50	.171	.608	2225
3/4"—S	7	1/8	2@1/12	3@1/8	4.50	.286	.763	4.50	.136	.503	2225
13/16"—R	5	1/8	3/16	2@3/16	5.25	.365	.898	4.50	.171	.608	2375
13/16"—R	7	1/8	2@1/12	3@1/8	5.25	.401	.988	4.50	.136	.503	2375
13/16"—S	7	1/8	2@1/8	3@1/8	5.25	.343	.845	4.50	.193	.617	2375
7/8"—R	7	1/8	2@1/8	3@1/8	6.00	.477	1.090	4.50	.193	.617	2600
7/8"—S	7	1/8	2@5/32	3@1/8	6.00	.427	.976	4.50	.243	.707	2600
15/16"—R	7	1/8	2@5/32	3@1/8	6.75	.581	1.241	4.50	.243	.707	2800
15/16"—S	7	1/8	2@3/16	3@1/8	6.75	.525	1.120	4.50	.299	.797	2800
1" —R	7	1/8	2@3/16	3@1/8	7.50	.701	1.402	4.50	.299	.797	3000
1" —S	7	1/8	2@1/8	3@3/16	5.25	.540	1.080	6.75	.460	1.131	3000
1-1/16"—R	7	1/8	2@1/8	3@3/16	6.00	.740	1.393	6.75	.460	1.131	3175
1-1/16"—S	7	1/8	2@1/6	3@3/16	6.00	.615	1.157	6.75	585	1.305	3175
1-1/8"—R	7	1/8	2@1/6	3@3/16	6.75	.839	1.490	6.75	.585	1.305	3350
1-1/8"—S	7	1/8	2@3/16	3@3/16	6.75	.771	1.371	6.75	.653	1.395	3350
1-3/16"—R	7	1/8	2@3/16	3@3/16	7.50	1.022	1.725	6.75	.653	1.395	3525
1-3/16"—S	7	1/8	2@7/32	3@3/16	7.50	.912	1.538	6.75	.763	1.526	3525

[1]Rough; [2]Sanded; [3]Refers to direction of face grain; *For sanded panels, thickness is before sanding.



Plywood girders, averaging 48" deep, spanning 36 ft., in R.C.A. Mfg. Co.'s Warehouse at Camden, N. J., designed and built by H. K. Ferguson Co. In all 198 identical girders were used, consisting of two ⅝" plywood webs, nailed to lumber flanges and stiffeners

Table 4

RECOMMENDED WORKING STRESSES FOR PLYWOOD (DOUGLAS FIR)

(For grades and thicknesses listed in US CS45-48)

In Bending, Tension and Compression (except bearing and 45° stresses) consider only those plies with their grain direction // to the principal stress.

DRY LOCATION

TYPE OF STRESS	EXT-DFPA•A-A	EXT-DFPA•A-B EXT-DFPA•Plyshield•(A-C)	EXT-DFPA•Utility•(B-C) EXT-DFPA•Sheathing•(C-C) EXT-DFPA•Concrete Form•(B-B) Plyform•(B-B) Plyscord•(C-D)	Interior•A-A Interior•A-B Plypanel•(A-D) Plybase•(B-D) NOTE: Apply the following percentages to the stresses for the corresponding Exterior grade. See example at bottom of page.
EXTREME FIBER in bending				
Face grain // to span	2,188	2,000	1,875	100%
" ⊥ "	1,875	1,875	1,875	80%
TENSION				
// to face grain (3-ply only*)	2,188	2,000	1,875	100%***
⊥ to face grain	1,875	1,875	1,875	80%
± 45° to face grain	337	320	310	85%
COMPRESSION				
// to face grain (3-ply only*)	1,605	1,460	1,375	100%***
⊥ to face grain	1,375	1,375	1,375	70%
± 45° to face grain	496	472	460	80%
BEARING (on face)	405	405	405	100%
SHEAR, rolling, in plane of plies**				
// or ⊥ to face grain	79	72	68	75%
± 45°	105	96	90	75%
SHEAR, in plane ⊥ to plies**				
// or ⊥ to face grain	210	192	180	85%
± 45°	420	384	360	85%
MODULUS OF ELASTICITY in bending				
Face grain // to span	1,600,000	1,600,000	1,600,000	100%
" ⊥ "	1,600,000	1,600,000	1,600,000	70%

*For tension or compression, // to grain, in 5-ply or thicker, use values for 3-ply, but in next lower grade.

**For certain conditions where stress concentrations exist these working stresses for rolling shear should be reduced by 50%. See Table I, F.P.L. Bulletin, "Approximate Methods of Calculating the Strength of Plywood."

***For 5 or more plies use 90%.

WET OR DAMP LOCATION

Where moisture content will exceed 16%, decrease by 20% values shown for Dry Location for following properties:

Extreme fiber in bending, tension and compression both // and ⊥ to the grain and at 45°, and bearing. (No change in values for shear or modulus of elasticity.)

Example: The working stress in compression // for Plypanel 5-ply (1238 psi) is found by multiplying the value for EXT-DFPA•Plyshield 5-ply, 1375 psi, by 90%, the reduction factor shown in the last column and footnotes * and ***.

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SECTION THREE

(Revised November 1, 1948)

PREVENTING CONDENSATION IN WALLS AND CEILINGS

MOISTURE CONDENSATION:

During the past decade, home builders have been confronted with a new problem, the presence of condensed moisture within exterior walls and roofs and under floors of dwellings. This moisture may manifest itself either by seeping through walls and ceilings to stain and damage interior decorations, by blisters and paint failures on exterior walls, and decay or even structural deterioration of the floor in basementless houses.

Paradoxically, certain improvements in construction and desire for greater comfort have served to emphasize condensation troubles. Automatic humidifiers increase the water vapor indoors; weather-stripping and tighter wall construction prevent air leaks and decrease heat losses, but they also retard vapor escape; insulation materials keep the outer wall sheathing colder by shutting it off from the inside heat.

Such improvements need not be cast aside, however, in order to check condensation. When the factors that contribute to it are understood, the remedies appear fairly obvious and simple.

Condensation is caused, during cold weather, by water vapor in the interior of a house passing through the wall lining and condensing as drops of water on the cold inner surface of the sheathing, building paper, siding, or even paint. Physicists explain this about as follows:

Some moisture is always present in the air in the form of water vapor which of course is a gas. Now, if air at any given temperature is loaded to capacity with water vapor, it is said to be saturated, i.e., the relative humidity is 100%. If the air contains only half of the water vapor it could hold, its humidity is 50%. Relative humidity therefore is simply a percentage showing the amount of water vapor present in the air, compared to the total amount of water vapor which the air might hold at the same temperature. The warmer the air, however, the greater the amount of water vapor it can contain, and conversely, as air is cooled, its capacity to hold vapor decreases.

Saturated air (i.e., air at 100% humidity) at 70° F., for example, contains about eight grains of water vapor per cubic foot, but when cooled to 31° F., only about two grains are needed to saturate the same volume; the other six grains will have been condensed into moisture. Furthermore, this same air, when saturated at 0° F., will contain less than one-half a grain.

The pressure of the water vapor is another influencing factor, and is the basic cause of vapor movement through walls. Vapor pressure varies directly with humidity, when the temperature remains the same. When room temperatures are the same, the vapor pressure at 50% relative humidity, for example, will be double that at 25% humidity.

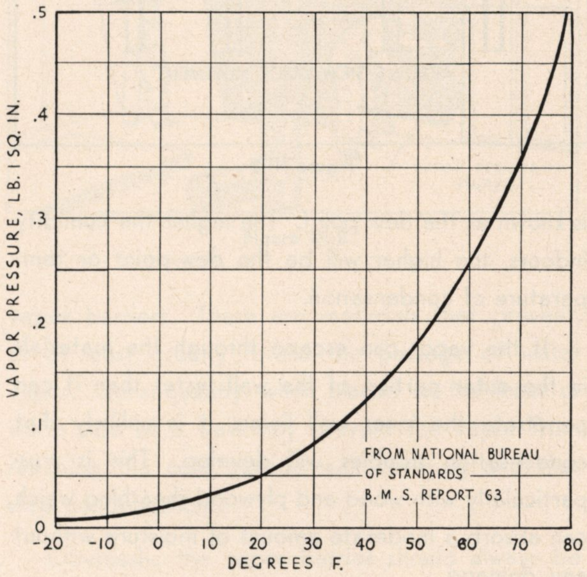

Figure III-1
Vapor Pressure at 100% Humidity

But at any given humidity, the vapor pressure in warm air is much higher than in cold air, as illustrated in figure III-1. It will be seen that the saturated vapor pressure at 70° is 0.36 lbs. per sq. in., while at 0° F. it is only 0.018, or one-twentieth as much.

Now even if the humidity is only 30% indoors and 80% outdoors, with respective temperatures at 70° F. and 0° F., the vapor pressure within will be approximately 0.100 lbs./sq. in., but only 0.015 lbs./sq. in. outside, or about 1/7 as much.

In other words, almost always during cold weather, water vapor inside is under constant pressure to move outwards and to pass through the inside wall lining, across the air space, and finally through the outside wall lining. This vapor movement may be likened to gas escaping from a balloon in which the rubber is slightly porous.

Now, if the vapor as it escapes outward becomes chilled sufficiently against the inner surfaces of the sheathing, for example, some moisture will condense and be deposited on that surface (see fig. III-2). The temperature at which this will occur is known as the dew point. The higher the humidity indoors, the higher will be the dew-point or temperature of condensation.

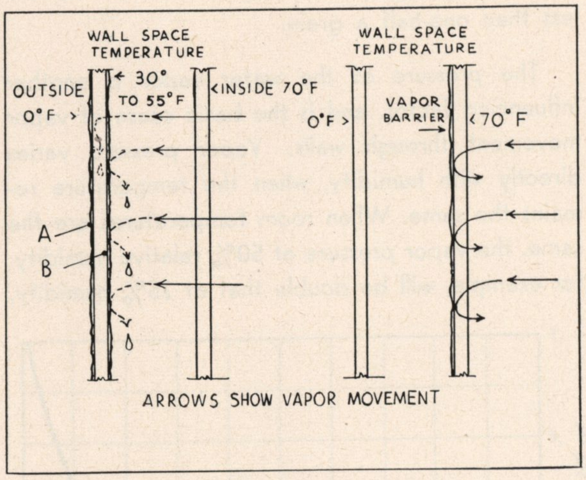

Figure III-2

If the vapor can escape through the materials in the **outer** portion of the wall faster than it can penetrate the inner wall lining, it is unlikely that condensation troubles will develop. This is true particularly with wood and plywood sheathing which can absorb a moderate amount of moisture without any damage.

Serious difficulties may result, however, when the reverse is true and the vapor seeps into the wall cavities so rapidly that instead of escaping to the outside, it keeps condensing.

One way to visualize this is to consider the wall as a basin, with an open water faucet and drain. If water enters faster than it can drain off, the basin overflows. Similarly, if water vapor enters a wall from the room-side faster than it can escape to the outside, there will be an "overflow" condition and condensation will occur whenever the outer wall lining is at a temperature below the dew-point.

PREVENTING CONDENSATION:

Humidity Control:

Low humidities indoors during the winter will minimize, if not entirely prevent condensation in walls and under roofs. Many houses however, have humidifiers built in the heating plant and people, generally recognizing the desirability of moderately high humidities, up to perhaps 40%, do not like their houses too "dry."

Further, water vapor is generated by laundry work, cooking (especially with gas), baths and by the body. In a small house, naturally there will be higher humidities than in a large house, other conditions being the same.

In general, therefore, the control of humidity is limited and additional steps must be considered in cold climates, if condensation is to be prevented.

Ventilation:

Probably the simplest way to prevent condensation in exterior walls, or under roofs, is to provide adequate ventilation by means of louvres, fans, open windows, etc., which permit the excess vapor within a building to escape readily by following the path of least resistance. This method will cause some heat loss, of course, but it should be kept in mind, that the prevention of similar heat losses, through cracks around doors and windows and through leaky, porous walls, accentuated the condensation problem when weather-stripping, for instance, was introduced. In other words, some attention to ventilation as a preventive of condensation appears desirable in any well-planned house or building and becomes imperative where high humidities exist indoors. It is better, however, to rely on definite controlled ventilation, than to trust to the vagaries of leaky construction.

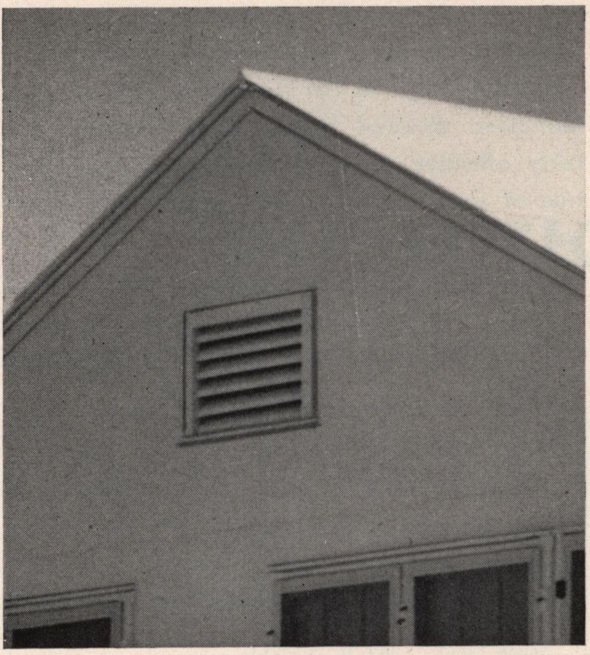

Figure III-3

Attic ventilation is desirable in most houses and is usually provided by means of screened louvres at gable ends (See photo III-3). The louvre in each gable end should have at least 1 square foot of opening for every 150 to 300 square feet of attic floor area. Still further ventilation is often furnished by a cornice design similar to figure III-4.

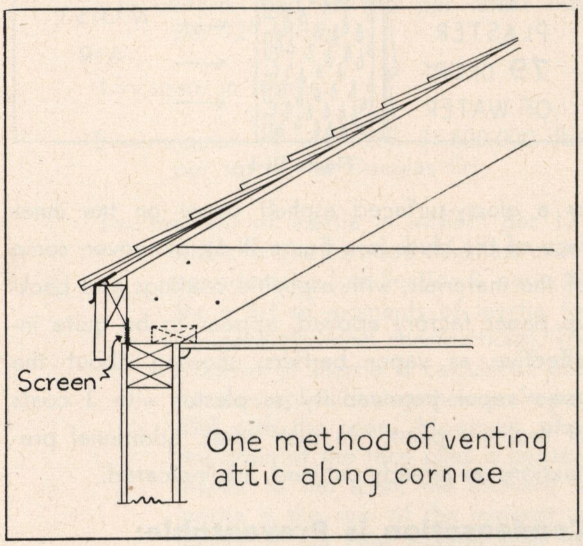

Figure III-4

Ventilators, consisting of adjustable grills or louvres, or small electric fans placed in the outside wall near the ceiling, may be desirable where high inside humidities are anticipated, especially for one-story houses in cold climates. About two square feet of grill area should be ample, even in moist climates. For unusual humidity conditions inside, an intake ventilator near the floor may be desirable in addition to the outlet vent. Lack of such ventilation caused grief in at least one recent instance, although other circumstances, such as use of gas stoves for heating, contributed greatly.

Openings from the outer wall cavities to the outside air have been advocated as a means of venting moist air which has entered the wall. This requires openings through the sheathing and siding or through the top plate, between each pair of studs. Such ventilation to be effective, would involve considerable air circulation in the stud spaces and would appear to nullify the insulation value of the outer wall lining and air space.

Vapor Barriers:

Equally important with ventilation as a means of eliminating condensation woes is the use of vapor barriers. These are materials that prevent, or at least greatly retard the passage of water vapor. Consequently, when placed on the inner or warm side of a wall they prevent the vapor from reaching the cold outside wall covering and thus stop condensation.

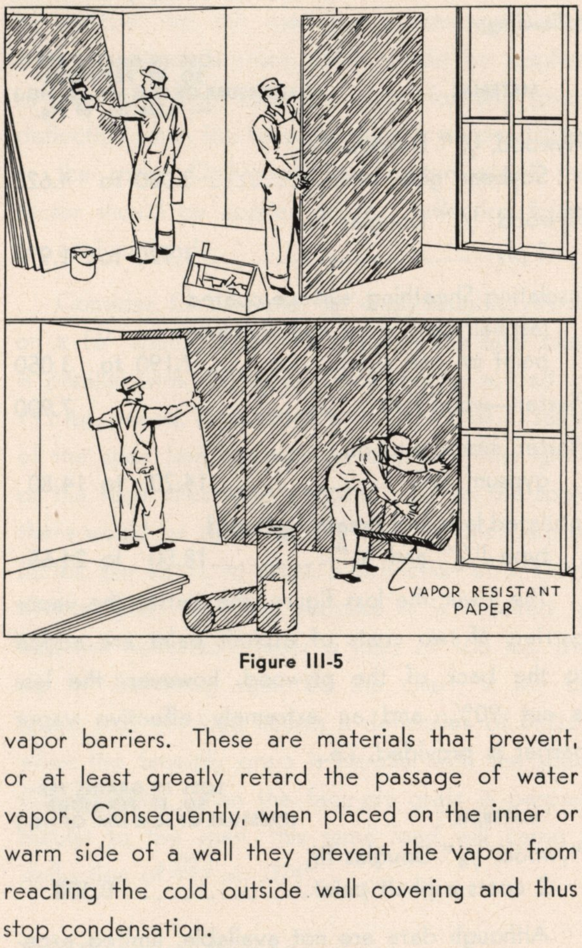

Figure III-5

Obviously, the vapor barrier should always be used on the inner or warm side on an exterior wall (see figure III-5) and not on the cold side; on the warm side, the vapor barrier serves to "close the faucet" whereas on the cold side it only plugs up the "drain" and invites condensation.

Examples of "plugged drain" construction are vapor-proof building paper between sheathing and siding, vapor proof materials used as sheathing, and high grade paint jobs that form a film over wood siding or plywood. "Poor" paint and "green" wood are often cited as causes of paint blisters that are actually caused by condensation.

The vapor-proof paper and sheathing may be avoided, of course, but obviously it would be impractical to use a poor paint job as a condensation preventive. It is far better to ventilate the house or to place a vapor barrier on the warm side of the wall.

Kinds of Vapor Barriers:

Materials such as plywood, plaster, and pulpboard, in common use for interior walls and ceilings are ineffective alone as vapor barriers. Tests at the U. S. Forest Products Laboratory show the following:

MATERIAL	LOSS IN GRAINS PER SQ. FT. PER HOUR (PRESSURE OF 18.45 MM. OF HG.) = 0.726 ins. of Hg.
Plywood, ¼″ Douglas fir, Soybean glue, plain	3.080 to 4.620
Plywood, ½″ Douglas fir, 5-ply	1.920 to 1.975
Insulating Sheathing, surface coated (asphalt both sides and aluminum paint on one side)	2.190 to 3.050
Plaster—wood lath	7.900
Plaster, fibreboard or gypsum lath	14.20 to 14.80
Insulated lath and sheathing-board type (½″ and ¾″)	18.50 to 24.65

The lower the loss figure, the better the vapor barrier. If two coats of asphalt paint are added to the back of the plywood, however, the loss is cut 90%, and an extremely effective vapor barrier is provided, i.e.,

MATERIAL	LOSS IN GRAINS PER SQ. FT. PER HOUR (PRESSURE OF 18.45 MM. OF HG.)
Plywood, ¼″ Douglas fir, 2 coats asphalt paint	0.308

Although data are not available, limited experience indicates that the vapor loss for an effective barrier should not exceed from 0.60 to about 1.00 grains under conditions noted in table.

An excellent vapor barrier for any wall material, is a glossy-surfaced asphalt-impregnated building paper, applied on inside face of studs. A coated paper is factory-applied on most of the batt-type insulations, designed to fit between studs or joists and widely used in the permanent type of prefabricated plywood house. Some of these are fairly effective as vapor barriers. The same is true of some of the asphalt coatings applied to wall materials.

Published test results,* however, indicate that none of these combination materials is as effective either as two coats of asphalt paint on plywood or a glossy-surfaced asphalt paper on the inner face of the studs (see figure III-6); moreover, some of the materials, with asphaltic coatings and back-up paper factory applied, appear to be quite ineffective as vapor barriers, showing about the same vapor permeability as plaster with 3 coats of ordinary paint. In such cases, additional precautions ,as already outlined, are indicated.

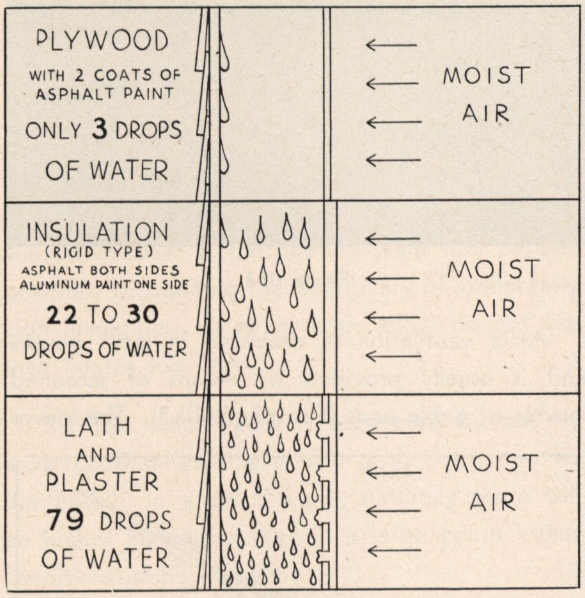

Figure III-6

Condensation is Preventable:

If the condensation threat is recognized, and the causes understood, it can be averted through proper design and construction.

Inside humidities may be partially controlled as a step in the right direction, but an effective vapor barrier on the warm side of walls and ceilings, supplemented by adequate and regulated ventilation, offers a simple and inexpensive answer to the condensation question.

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*See Bureau of Standards BMS 63 and U. S. Forest Products Laboratory, "Condensation Problems in Buildings."

SECTION FOUR

Revised November 1, 1948

DEFLECTION CHARTS

INTRODUCTION

The following charts represent graphically the deflection of Douglas fir plywood under a uniformly distributed load when considered as a simple beam resting on two supports. (When the plywood is continuous over more than two supports, the deflection will be considerably less for a given load as explained in a later paragraph.) Each line on the chart is a solution of the equation,

$$d = \frac{5}{384} \frac{wl^4}{EI}$$

where $d =$ deflection, in inches;

$w =$ uniform load, in lbs. per linear inch per 12″ width;

$l =$ span, in inches;

$E =$ modulus of elasticity, (1,600,000 lbs. per sq. in. for Douglas fir);

$I =$ moment of inertia in inches^4 per 12″ width for the plywood thickness under consideration (see Section 2, page 10, for a list of moments of inertia for standard plywood constructions). The moment of inertia is calculated only for those plies having their grain parallel with the span. However, when the grain of the face plies is perpendicular to the span, the moment of inertia is the sum of the moment of inertia of the plies parallel with the span plus 1/20th of the moment of inertia of the perpendicular plies. This method of computation is explained and illustrated in Section 2, Page 1.

By means of the formula above, the deflection of plywood under a given load may be found for any intermediate span or construction not shown in the charts.

The spans for which deflections are shown range from 12″ to 24″ and are those commonly used in construction. They represent the distance, center to center, of the supporting members.

Direction of Face-Ply Grain

The charts are for plywood placed so that the grain of the face plies is parallel with the span, or perpendicular to the supports. This is the usual method of construction, as the plywood is both stronger and stiffer than it would be if used the other way. However, under certain conditions, plywood panels are placed so that the grain of the face plies is perpendicular to the span, in which case the deflection under a given load will be greater; for this reason a percentage factor is shown on each chart which should be applied to the load read from the chart for any given deflection when the face-ply grain is perpendicular to the span. Conversely, the reciprocal of the factor should be applied to the deflection found for a given load.

Consider, for example, 5/8″ 5-ply rough plywood on a 16″ span. When the grain of the face plies is parallel with the span, it will require a load of 193 lbs. per sq. ft. to cause a deflection of 1/360th of the span (see appropriate chart). If the grain of the face plies were perpendicular to the span, there would be required only 31% of this load, or 60 lbs. per sq. ft. to cause a deflection of 1/360th of the span. Similarly, suppose it is required to find the deflection under a load of 100 lbs. per sq. ft. Referring again to the chart, it will be found that this load causes a deflection of 0.023″ when the face-ply grain is parallel with the span. Consequently, when the face-ply grain is perpendicular to the span, this same load will cause a deflection of $\frac{.023}{.31} = .074''$.

Continuous Spans

Ordinarily, plywood is used, not on a single span, but over a multiple number of equal spans, in which case it functions as a continuous beam. An example of this would be an eight-foot panel of plywood placed on joists at 16″ centers, in which case the plywood may be considered as a six-span continuous beam. Not only will the deflection of each span be considerably less than for a simple beam of the same span for the same load, but the stresses will also be reduced in most cases. The maximum deflection in a continuous beam of equal spans and under uniform load occurs in the outer span and is approximately one-half of the deflection of a simple beam of the same span and load. Consequently, it requires twice as much load to produce a given deflection in a continuous beam as it does to produce an equal deflection in a simple beam.

Consider again, 5/8″ 5-ply rough plywood as a simple beam on a 16″ span. The load required to cause a deflection of 1/360th of the span will be 193 lbs. per sq. ft. (see chart). However, if the plywood were continuous over two or more 16″ spans, the load required to cause the same deflection in the outer spans, i.e., 1/360th of the span, would be $2(193) = 386$ lbs. per sq. ft. Also, if the deflection of a simple beam caused by a load of 100 lbs. per sq. ft. is 0.023″, the same load would cause a deflection in the outer spans of only $\frac{.023}{2} = .0115''$ if the plywood were continuous.

Allowable Deflections

In many cases, the amount of load that can be placed on plywood acting as a beam will not be limited by the strength of the plywood, but rather by its stiffness as measured by the amount of deflection. Allowable deflection is usually expressed as a fraction of the span length, such as 1/360th, 1/270th, etc. In some instances the amount of deflection is governed by the circumstances of the construction; e.g., when plaster is used as a covering, the deflection of the supporting joists should not exceed 1/360th of the span, else the plaster is likely to crack. However, in dry-built construction, there is no plaster, and consequently a somewhat larger deflection, such as 1/270th of the span, may be allowed, unless it is otherwise limited by governing building laws. As an aid to determining the load that will cause a given deflection, dotted lines are shown on the charts for 1/360th and 1/270th of the span, the latter also being the fraction usually allowed for concrete forms in order to prevent noticeable irregularities in the face of the finished concrete.

Bending Stresses

Another dotted line shown on the charts indicates the load which will cause a bending stress in the plywood of 2,000 lbs. per sq. in., which applies only when the direction of the face-ply grain is parallel with the span. The stress is calculated trom the usual engineering formula for bending stress, except that a factor of 0.85 is included, i.e., $M = \frac{.85SI}{C}$. This should not be interpreted as being the allowable working stress for plywood, but only as an indication of the degree to which it will be stressed under a given load. Actually, allowable stresses in bending for Douglas fir plywood will vary from 1500 to 2500 lbs. per sq. in. depending upon the grade and the exposure conditions.

If the allowable working stress in bending or horizontal shear is exceeded by a loading caused by wet concrete, it should be noted that this is a short-time load, and consequently, the working stresses may be increased by 50%. (See Section 2 for a discussion of working stresses.)

Neglected Factors

The deflections listed in the following charts, are based on standard formulae, substantiated by hundreds of tests. There may be some minor variation between the theoretical values and the actual values which obtain in service, because of conventional construction procedure, the inherent variability in the characteristics of wood, and certain theoretical considerations whose effects are usually negligible.

Some of these factors which affect the actual deflection of plywood are plate action, which occurs when a plywood beam is supported at intervals along its width by members running parallel with the span, the nailing or gluing of the plywood to the supports, the addition of a finish flooring, and deflections due to shear. Under ordinary conditions the effects of these factors tend to cancel, with the result that values read from the deflection charts will closely represent the actual deflections usually encountered in construction.

THE DEFLECTION OF PLYWOOD CONCRETE FORMS

The stiffness of wood is decreased by an increase in moisture content and an increase in the duration of stress. Since both of these factors come into play when plywood is used for concrete forms, it is desirable to learn to what extent the stiffness of a plywood panel is affected by concrete loading in order to make an intelligent correction to deflections calculated by ordinary formulaes which employ a modulus of elasticity found from short-time tests on dry plywood.

When concrete is poured into a form, the form is subjected to wetting on its inside face, as well as to bending stress caused by the hydrostatic pressure of concrete. This pressure will continue at its maximum amount for perhaps as long as four hours, at which time the concrete will have achieved its final set and the loading will diminish in intensity.

In order to approximate this condition in the laboratory, the upper surface of a 4 x 8 panel of ½″ thick plywood was flooded with water for four hours, in order to bring its moisture content to that of a concrete form, then loaded uniformly to 86 lbs. per sq. ft., by means of sand bags. Deflection readings were taken at the end of four hours of loading. The panel was continuous over four spans of 24″ each, the readings being taken in the two outer spans. Comparison was then made with deflection values previously determined from a dry loading of the panel, the values in this test having been read immediately after loading.

This test was performed on three different ½″ plywood panels. For confirmation a similar test was carried out on a ½″ panel, four spans at 20″, four feet wide, loaded to 121 lbs. per sq. ft. with wet concrete (six gallons of water per sack of cement). In this test deflection ceased 3½ hours after pouring; the effect of the moisture was not evidenced until 1¼ hours after loading.

The results of the tests are shown below, the deflections being the average of the two end spans.

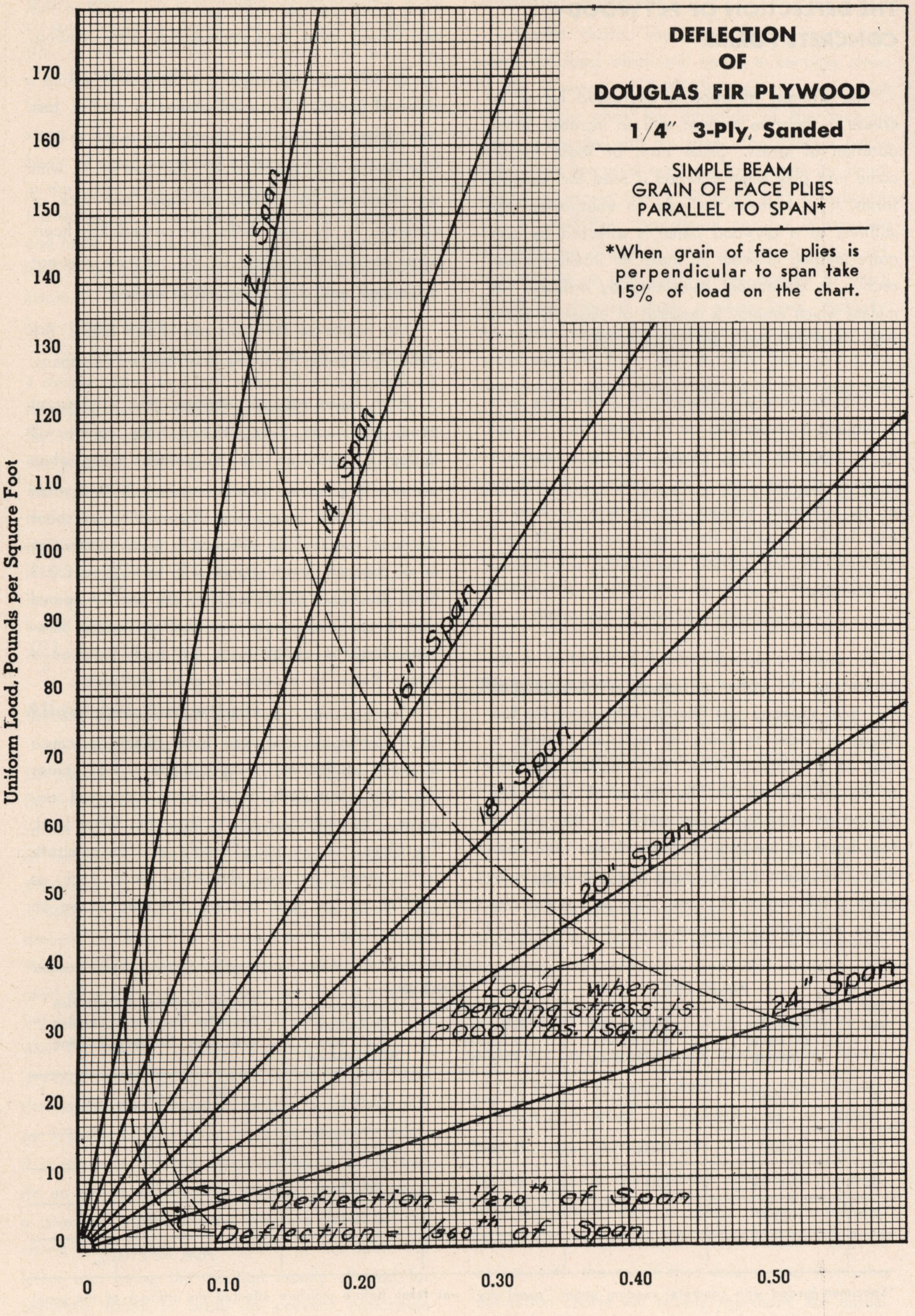
From these tests, it appears that a plywood panel continuous over two or more spans, will deflect approximately 20 percent more when wetted by concrete poured against it than when used dry. This means that for continuous spans the load required to produce a given deflection in a concrete form panel will be $\frac{100}{120} = 0.833$ times the load computed for dry use. A panel that is continuous over two or more spans, however, requires about twice as much load as a simple beam to produce a given deflection.

Consequently, the load that may be placed on a continuous concrete form panel, to cause a given deflection, $= 2 \times 0.833 = 1.67$ times the load computed for a simply supported dry panel. The deflection charts show this latter load. Therefore, to get the allowable load on concrete form panels, for any deflection limit, such as 1/270th of the span multiply the load shown on the chart by 1-2/3.

Test No.	PANEL DRY		PANEL WET		Percent Increase of Deflection for Wet Four-Hour Loading over Dry, No-Hour Loading
	Deflection at Time of Loading	Deflection Four Hours After Loading	Deflection at Time of Loading	Deflection Four Hours After Loading	
1.	0.122″	0.1235″	0.1445″	0.154″	$\frac{.154 - .122}{.122} = 26\%$
2.	0.130″	0.131″	0.144″	0.150″	15%
3.	0.1145″	0.117″	0.123″	0.134″	17%
4.*	.081″			0.1035″	28%
					Ave. = 21½%

*Specimen loaded with concrete; reading under "panel dry" was taken before moisture affected the stiffness of the panel.

DOUGLAS FIR PLYWOOD ASSOCIATION
Tacoma 2, Washington



DEFLECTION OF DOUGLAS FIR PLYWOOD

1/4" 3-Ply, Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

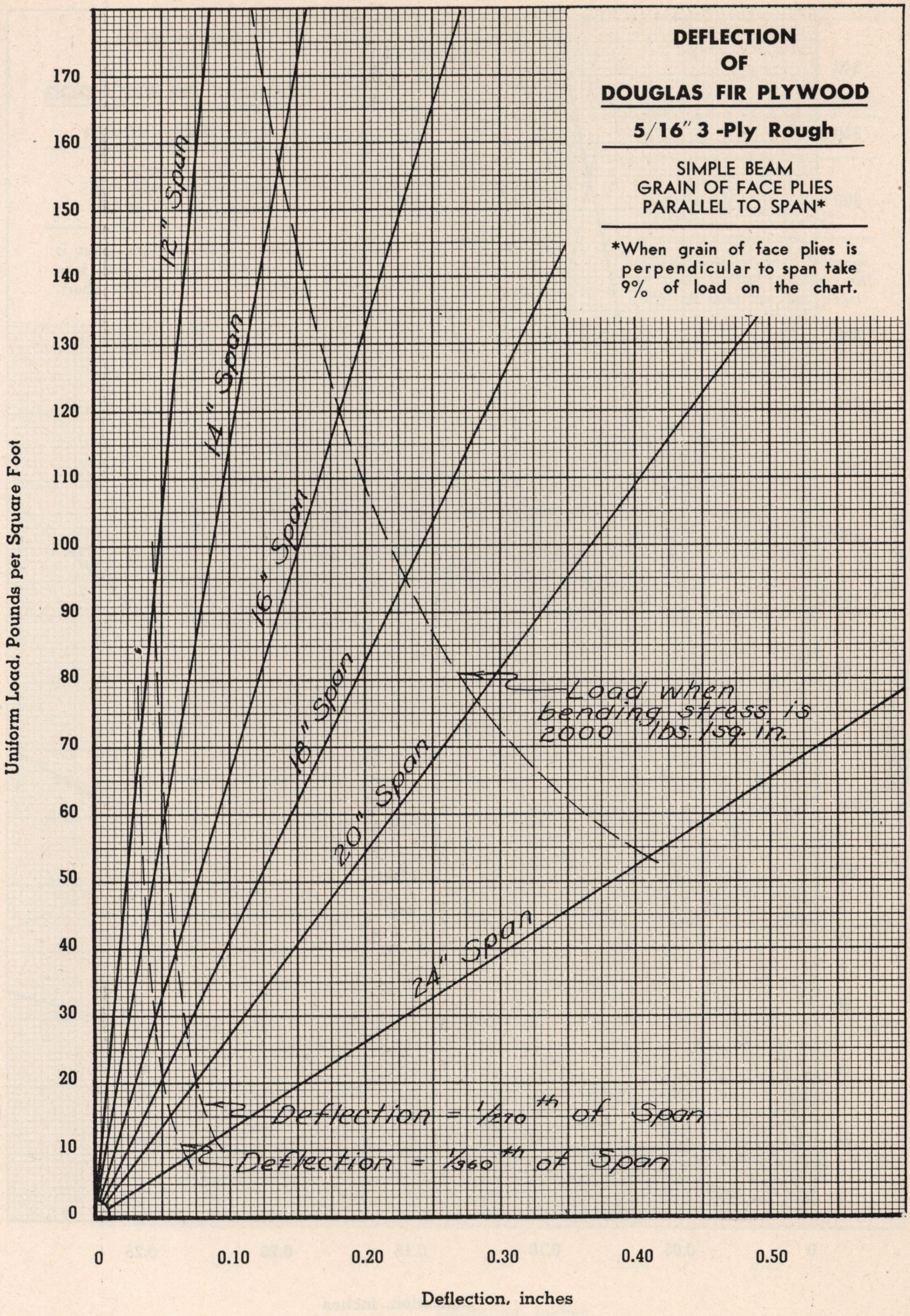
*When grain of face plies is perpendicular to span take 15% of load on the chart.

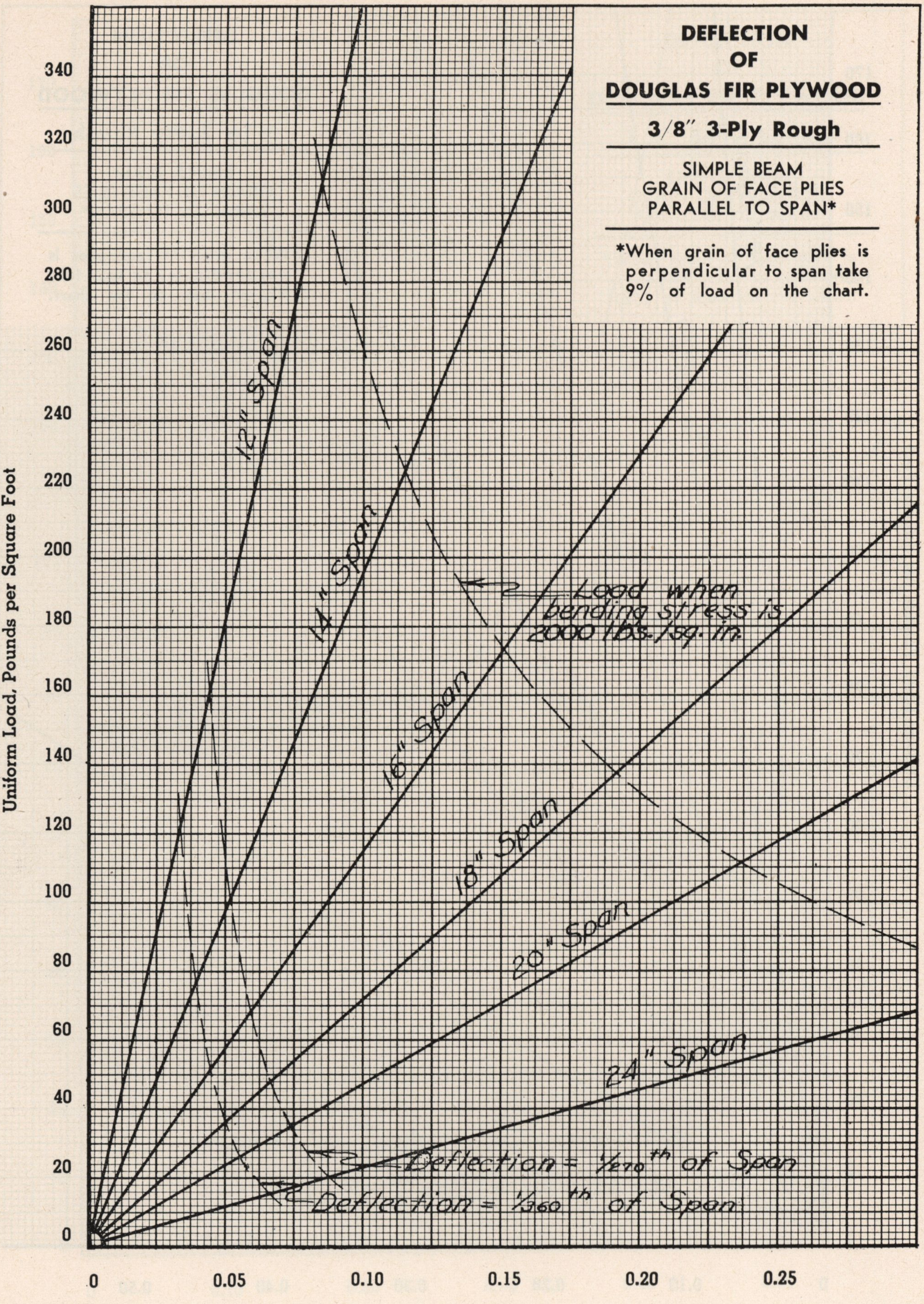
DEFLECTION OF DOUGLAS FIR PLYWOOD

5/16″ 3-Ply Rough

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 9% of load on the chart.



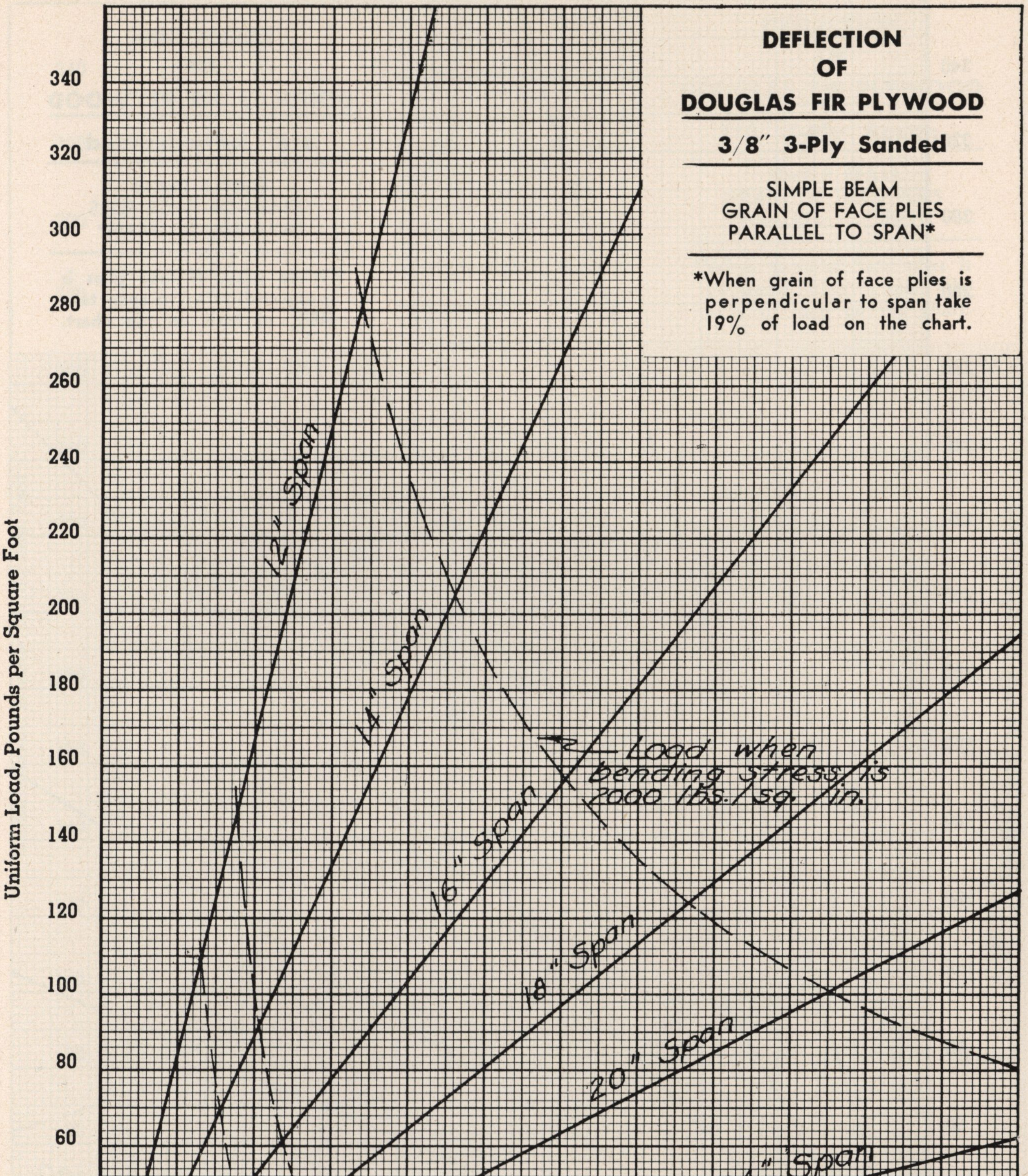


DEFLECTION OF DOUGLAS FIR PLYWOOD

3/8″ 3-Ply Rough

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 9% of load on the chart.

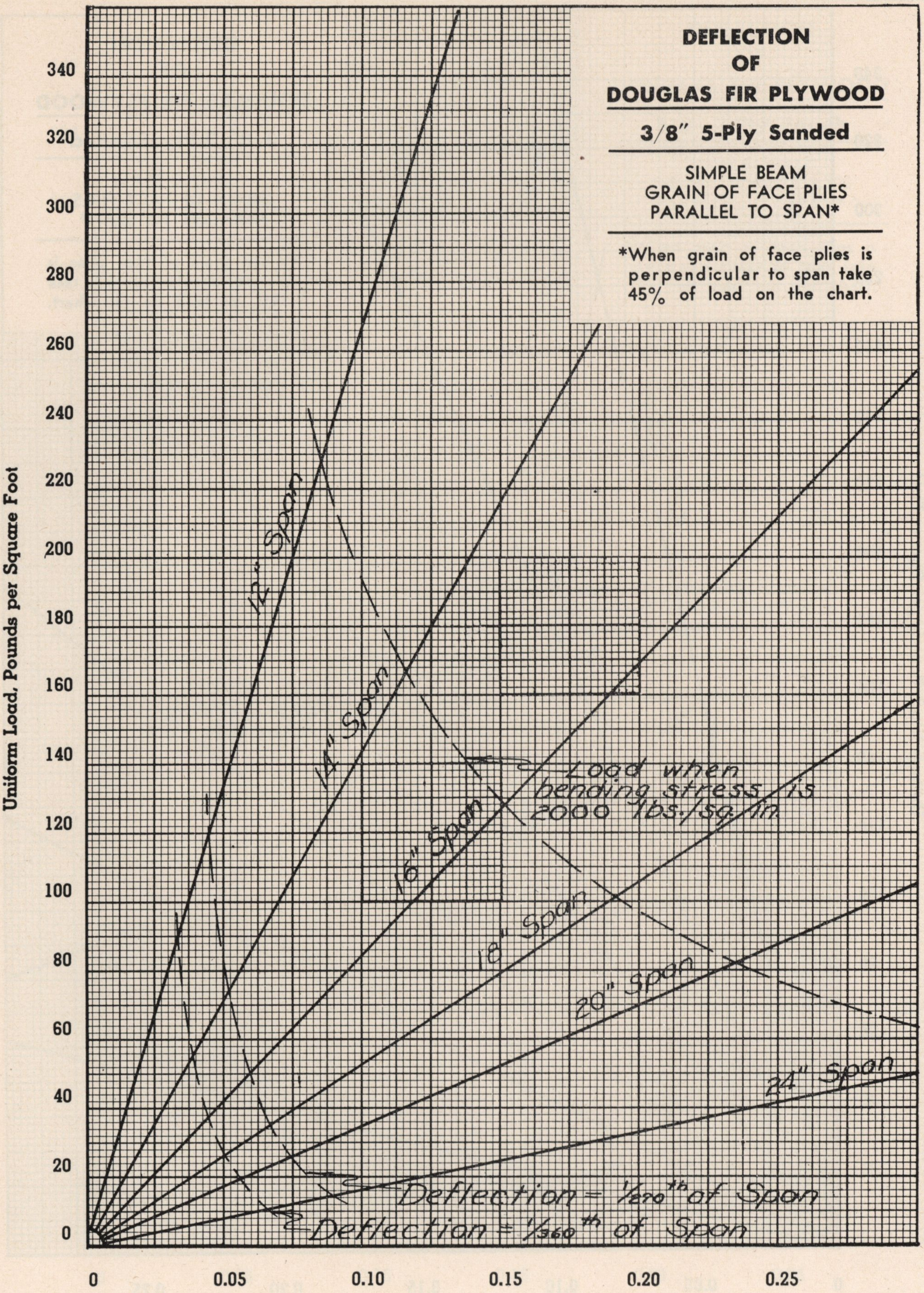


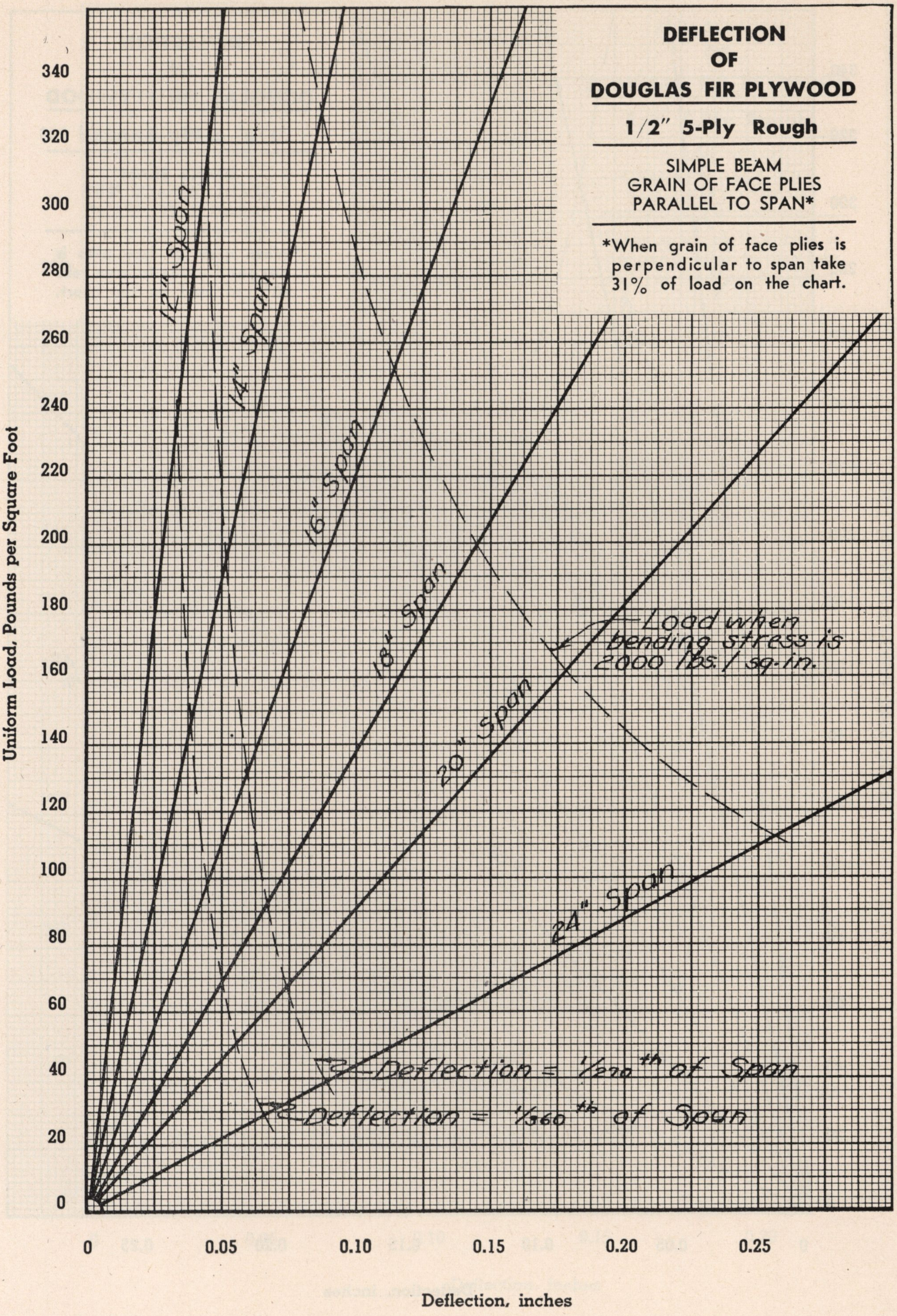
DEFLECTION OF DOUGLAS FIR PLYWOOD

3/8″ 3-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 19% of load on the chart.


DEFLECTION OF DOUGLAS FIR PLYWOOD
3/8" 5-Ply Sanded
SIMPLE BEAM
GRAIN OF FACE PLIES PARALLEL TO SPAN*
*When grain of face plies is perpendicular to span take 45% of load on the chart.
Uniform Load, Pounds per Square Foot
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
Deflection, inches
0
0.05
0.10
0.15
0.20
0.25
12" Span
14" Span
16" Span
18" Span
20" Span
24" Span
Load when bending stress is 2000 lbs./sq. in.
Deflection = 1/270th of Span
Deflection = 1/360th of Span

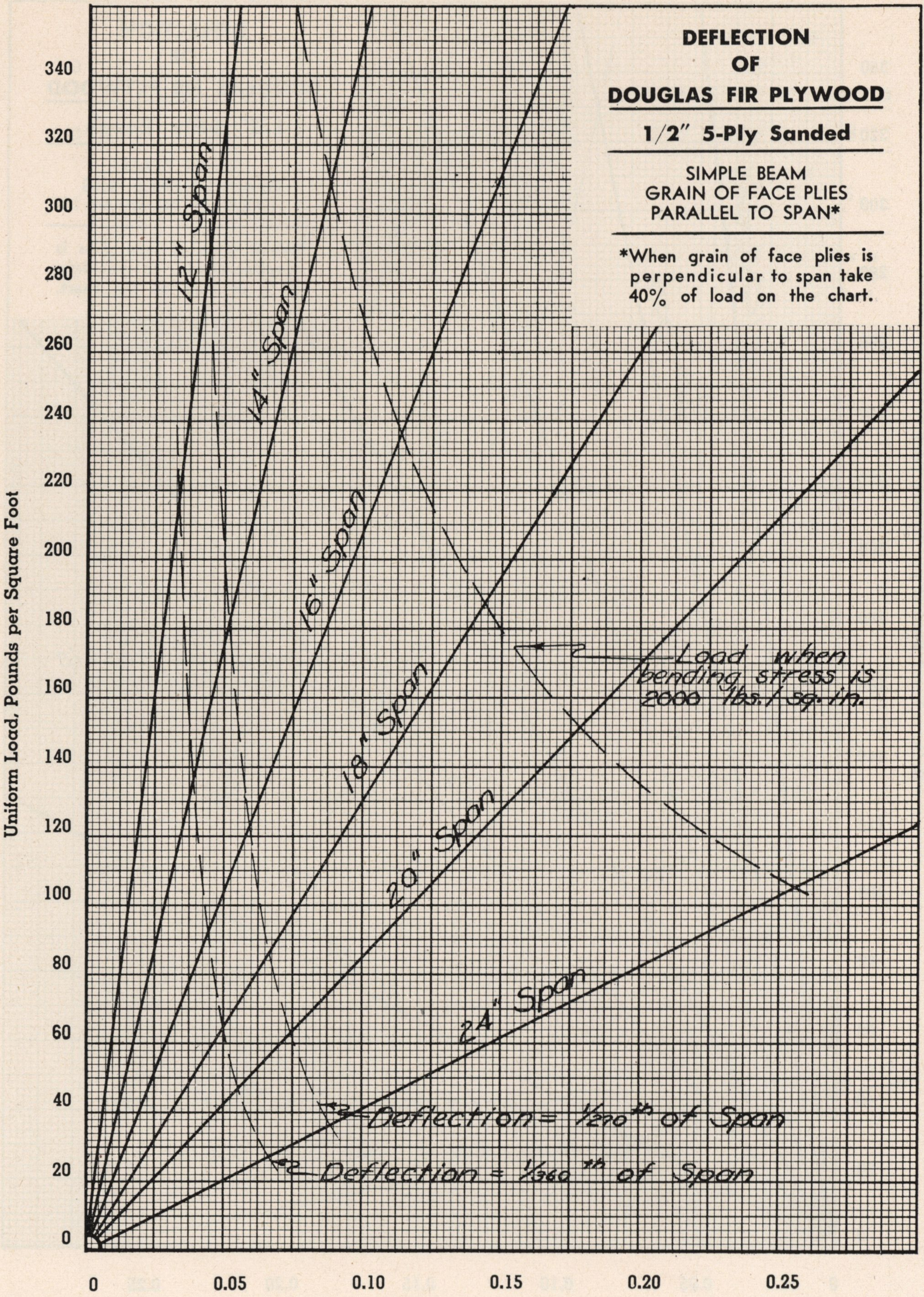


DEFLECTION OF DOUGLAS FIR PLYWOOD

1/2″ 5-Ply Rough

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 31% of load on the chart.

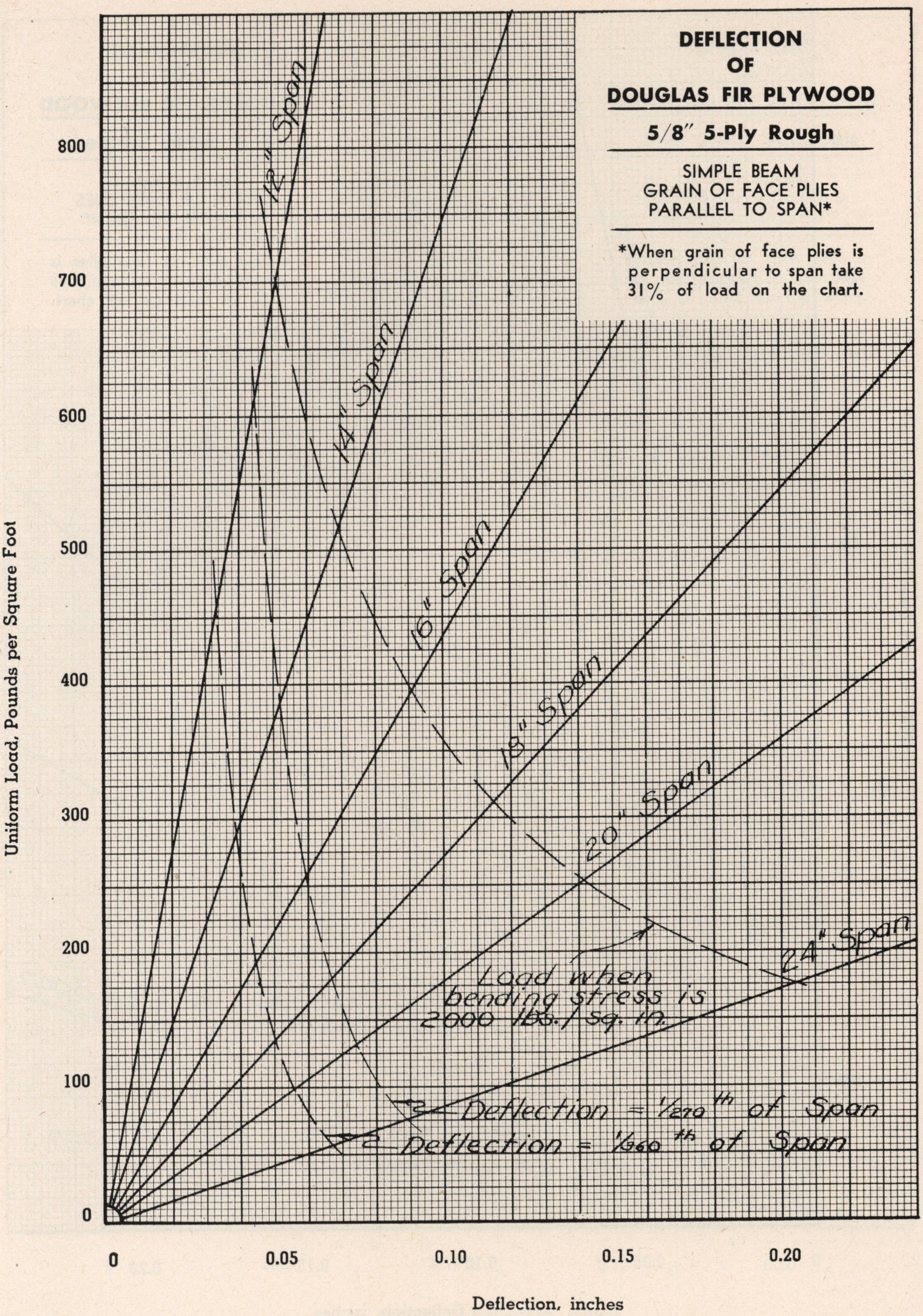


DEFLECTION OF DOUGLAS FIR PLYWOOD

1/2" 5-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 40% of load on the chart.

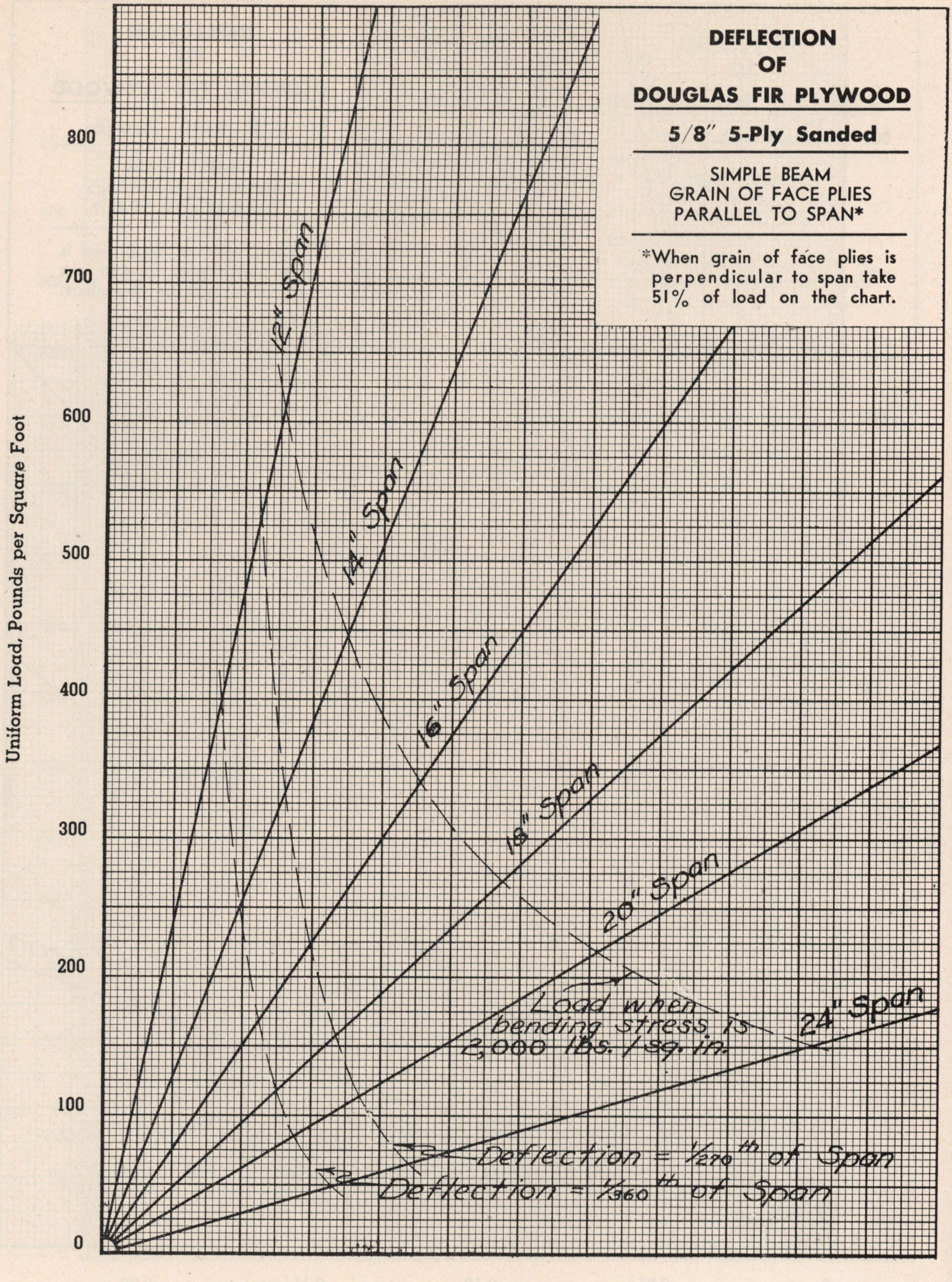


DEFLECTION OF DOUGLAS FIR PLYWOOD

5/8″ 5-Ply Rough

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 31% of load on the chart.

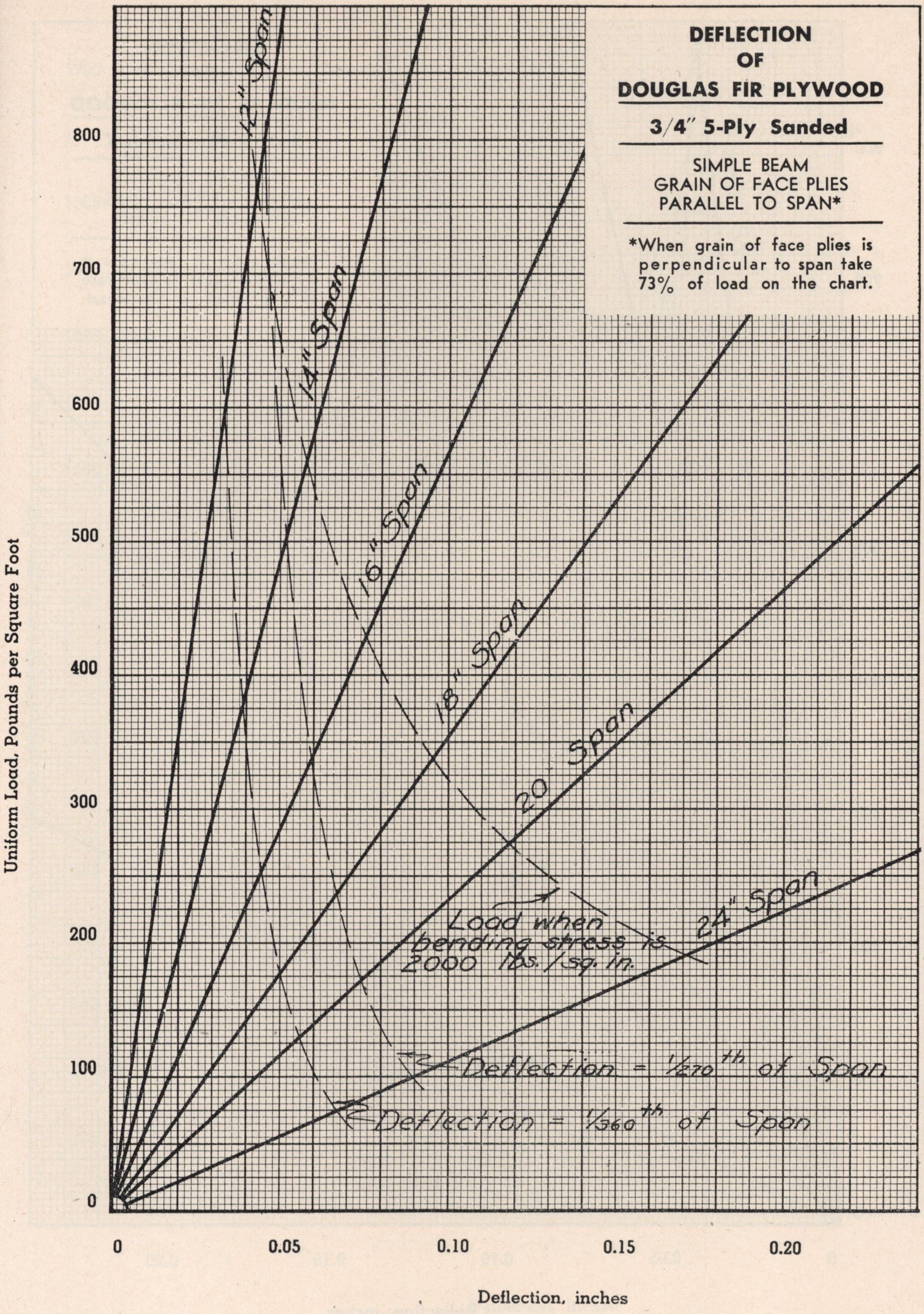


DEFLECTION OF DOUGLAS FIR PLYWOOD

5/8″ 5-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 51% of load on the chart.

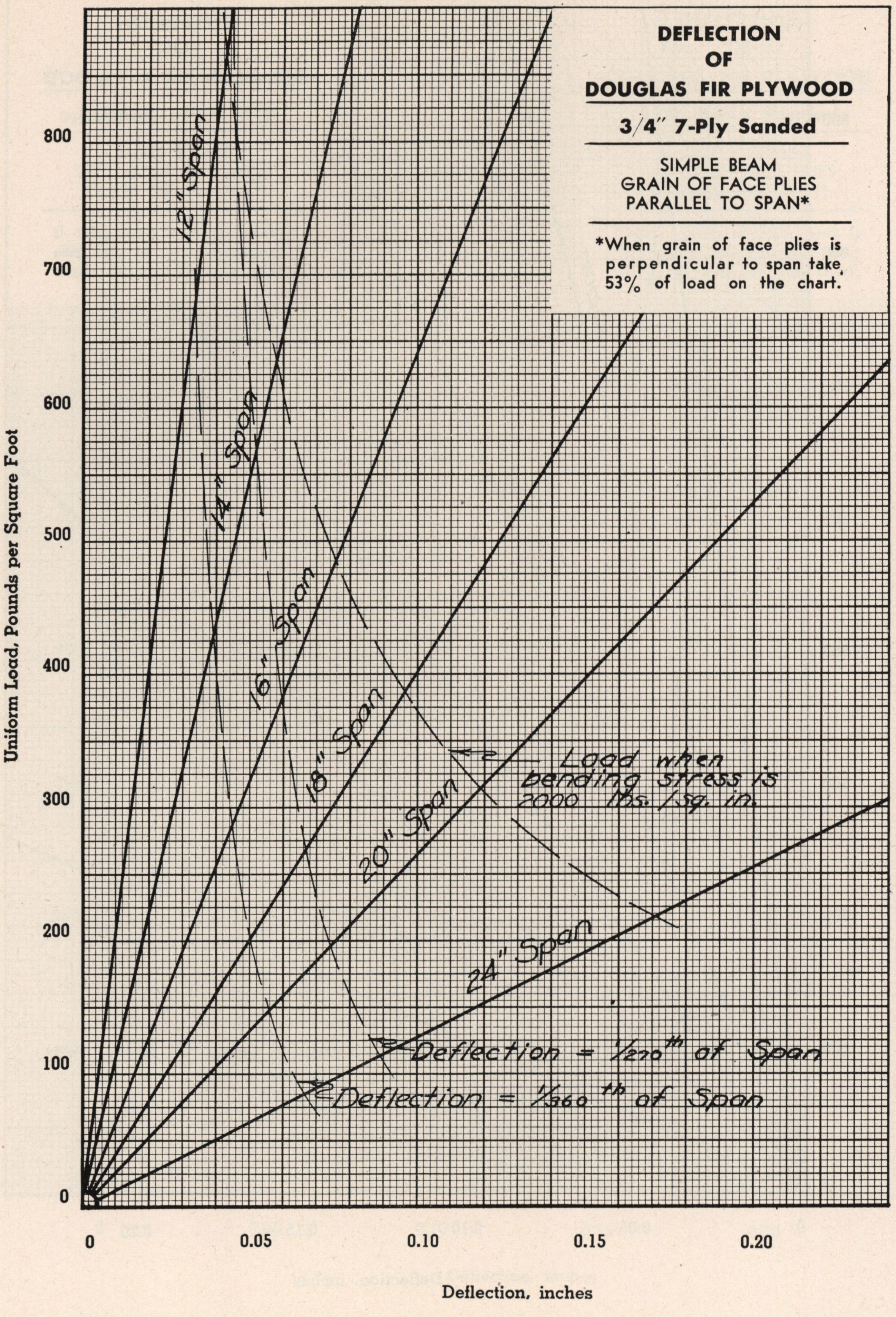


DEFLECTION OF DOUGLAS FIR PLYWOOD

3/4″ 5-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 73% of load on the chart.

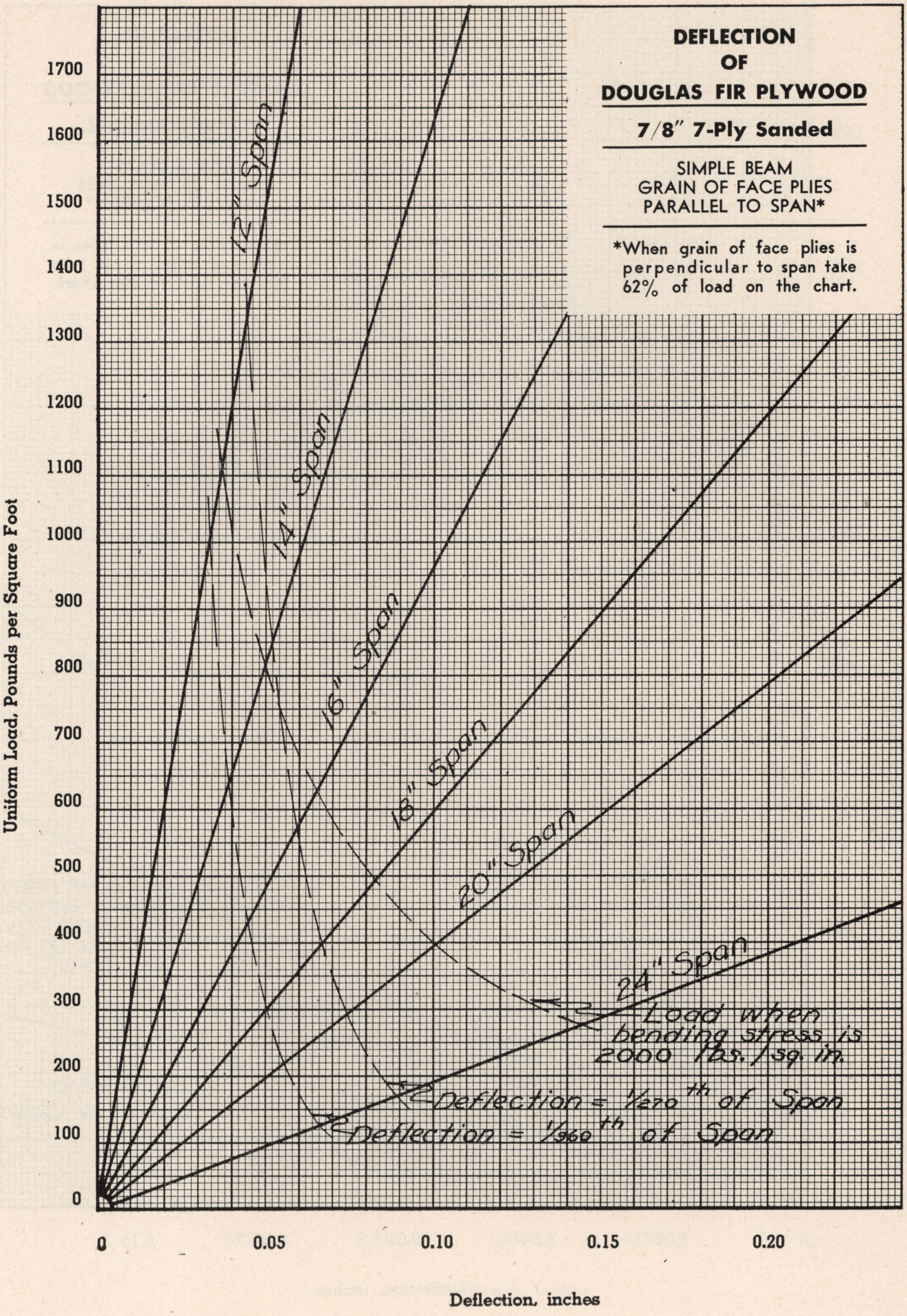


DEFLECTION OF DOUGLAS FIR PLYWOOD

3/4″ 7-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 53% of load on the chart.

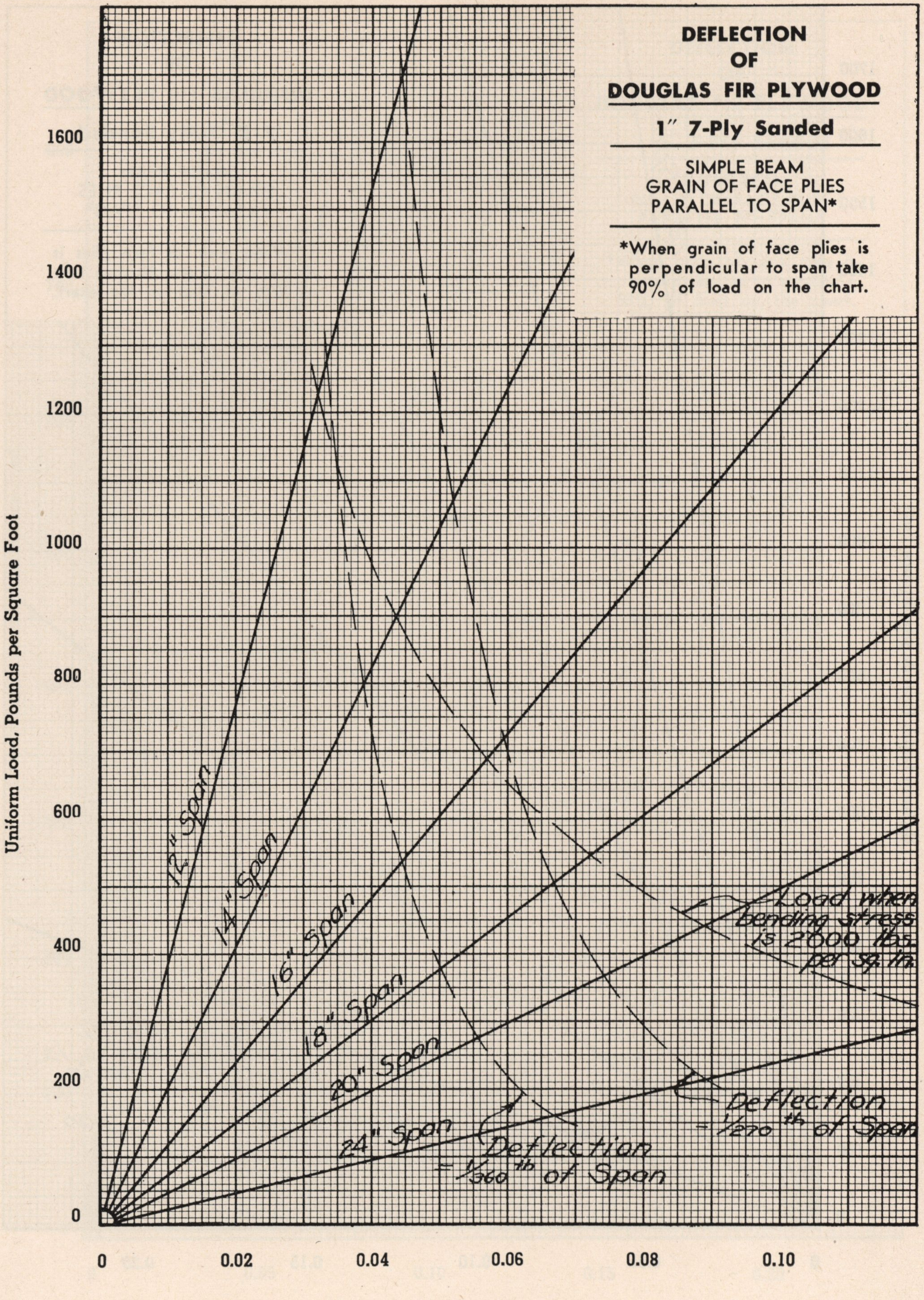


DEFLECTION OF DOUGLAS FIR PLYWOOD

7/8″ 7-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 62% of load on the chart.

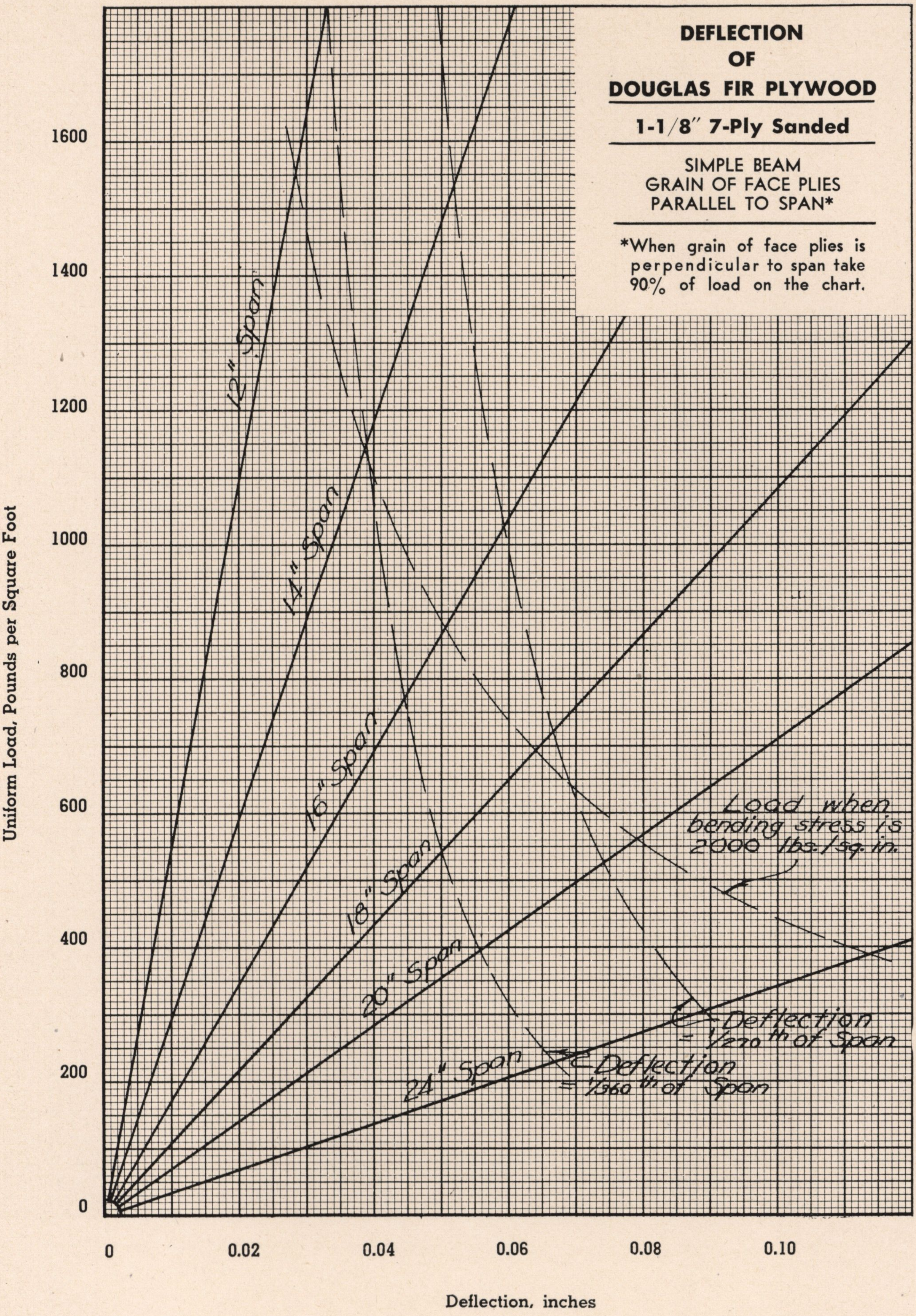


DEFLECTION OF DOUGLAS FIR PLYWOOD

1″ 7-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 90% of load on the chart.



DEFLECTION OF DOUGLAS FIR PLYWOOD

1-1/8″ 7-Ply Sanded

SIMPLE BEAM
GRAIN OF FACE PLIES
PARALLEL TO SPAN*

*When grain of face plies is perpendicular to span take 90% of load on the chart.

Revised November 1, 1948

INSULATING WITH PLYWOOD

This section is designed to provide data on the thermal insulation properties of plywood in wall, roof and floor sections, and to show the effect of adding special insulation where deemed desirable.

Heat losses from buildings occur through walls and roofs, windows and doors, to a lesser degree through floors, and, to a varied extent, as a result of air leakage and infiltration.

These heat losses are influenced by air currents conveying heat from warmer surfaces and zones to those of lower temperature; by heat radiating from one object to another of a lower temperature; and by conductance of heat through walls, roofs, glass, and the like, from the inside toward the outside of a building.

Moreover, high winds may blow cold air into a building and draw out warm air through porous or semi-porous materials.

In considering heat losses from a building the insulation value of the wall and roof construction is important, but it can be unduly emphasized.

Various estimates and calculations indicate that in an "average" house, not weatherstripped and not specially insulated, the heat loss will be about 30 per cent through the walls, about 30 per cent through the roof, about 35 per cent through and around windows and doors, and the remainder through first floor and basement.

It will be apparent, therefore, that even if the insulation value of walls and roof is doubled (i. e., if their resistance to heat transmission is cut in half), the total reduction in heat loss from the dwelling, due to such improved insulation, will be limited to about 30 per cent.* Such a saving is considerable, but generally requires adding special insulation in wall cavities rather than substitution of materials.

For example, one material, A, may be advocated over another, B, by reason of superior insulating properties, perhaps two or three times as great. The consumer might conclude that by using A, his heating bill would be divided by 2 or 3. Unfortunately this desirable result cannot be obtained so easily. The reason is that when material A is placed on the wall, its resistance is added to that of the other materials, the air space and the surfaces, so that the total resistance of the wall will be increased perhaps only 25 per cent.

For example, assume a wall with a U value (coefficient of thermal transmission) of .25 and a resistance of 4.00.

If the inner lining, with a resistance of 0.40 is replaced with a special material having a resistance of 1.40 (3½ times as great), the wall's resistance is raised to 5.00 and the U value becomes 0.20 B.T.U.'s. The net insulation gain is 20 per cent for the wall, or only a 6 per cent (20 per cent of 30 per cent) reduction in the heat loss from the entire house, a far cry from the 3½ to 1 ratio in the two wall lining materials.

Weatherstripping is especially effective in minimizing air-leakage around openings, and double glazing will greatly reduce heat loss through single sheets of glass. If a building has such protection, the total heat loss will occur mostly through walls and roof, and then added insulation causes a higher percentage reduction in fuel savings than in the unprotected house, but not, of course, any more actual savings.

SELECTION OF MATERIALS AND CONSTRUCTION TYPES:

Insulation value is but one factor in considering design and construction. Others are strength and rigidity, durability, resistance to abrasion and impact, appearance, finishing and cost.

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* i. e., ½ (30% for walls and 30% for roof.)

Nevertheless, certain minimum requirements, varying with climatic conditions, exist regarding insulating values. These values, if too low, may adversely affect body comfort, increase the condensation hazard, (see Section Three) and increase unduly the fuel bill.

Plywood construction can be designed to meet any of these requirements. The DFP (Prefabricated) Dri-Bilt Construction consisting of plywood wall lining inside and sheathing, furring strips and siding or shingles outside is practically identical in insulating value with the older "standard type" of siding, paper and sheathing, with lath and plaster on inside.

Simple panel construction using Exterior plywood on outside face of studs and plywood wall lining on inside, usually requires added insulation in the form of batts or blankets, fill or even reflective sheets in the wall cavities, except in mild, southern climates.

Insulation values for various plywood constructions, and the effect of substitutions, are listed in the tables on pages 4 to 8.

DETERMINING THE "U" VALUE OF WALLS AND ROOFS:

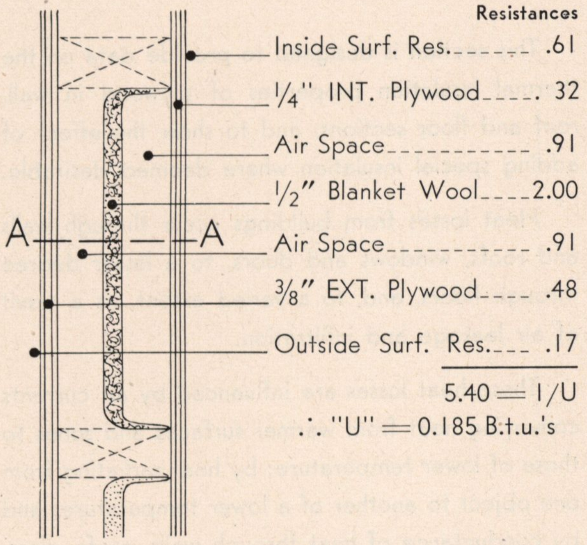
Numerous laboratory tests have established the **conductivity** or "k" value for various homogeneous building materials; it is simply the amount of heat, measured in B.T.U.'s transmitted through one inch of thickness, per square foot, per hour, for each degree temperature difference between the opposite surfaces.

For non-homogeneous materials such as hollow tile, and for materials not 1″ thick, the unit heat transfer is known as **conductance.**

Resistivity or resistance (1/k) is the reciprocal of conductivity or conductance, and varies directly with the thickness. It may be considered as the temperature difference needed between the two surfaces of a material, to transmit one B.T.U. per square foot per hour through the material.

Accordingly, for a wall built of known materials, it is possible to add up the resistance of the component parts, with due regard to thickness of each material used, as well as the resistances of any air spaces and of the inner and outer surfaces. This total wall resistance gives a numerical value which, as noted in previous paragraph, will be the degrees temperature difference needed betwen the air on opposite sides of the wall to transmit one B.T.U. for each square foot, every hour.

The reciprocal of this summation will be the "U value" or coefficient of thermal transmission of the wall. For example, see figure 1.



	Resistances
Inside Surf. Res.	.61
1/4" INT. Plywood	.32
Air Space	.91
1/2″ Blanket Wool	2.00
Air Space	.91
3/8″ EXT. Plywood	.48
Outside Surf. Res.	.17
	5.40 = 1/U

∴ "U" = 0.185 B.t.u.'s

Fig. 1

Typical resistances through insulated plywood wall.

AIR SPACES AND SURFACE RESISTANCE:

Two points may be of interest. First, the insulation value of still air in a wall space remains fairly constant for spaces ranging from 3/4″ to several inches in thickness. This covers the spaces generally encountered in wood-framed construction. Tests have established an average value of 0.91 for the resistance of the ordinary air space, at least 3/4″ deep. For lesser widths, the resistance to heat flow decreases and should be obtained from published tables. A 1/4″ space for example has a resistance of about 0.67.

Second, a surface resistance, considered as equal for all ordinary materials, is provided by a thin film of air lying immediately along the surfaces. Since wind action reduces this resistance, theoretical insulation values for a wall are generally based on an inside surface resistance (still air) of 0.61, whereas the outside surface resistance is taken as 0.17 for a 15 mile per hour wind. Both values have been obtained from accepted laboratory tests.

TEMPERATURE GRADIENT THROUGH WALL:

It is sometimes desirable to know the temperature of various parts of a wall, in order to check, for example, the likelihood of condensation.

The temperature drop through any wall may be obtained easily by simple proportion, knowing the resistance of the entire wall, and of its component parts.

In figure 1, along section A-A, for example, the wall resistance is 5.40, which is the sum of the resistances of the separate parts.

Now, if the air temperature inside is kept at 70° and outside at 0° F., the temperature drop through the entire wall is, of course, 70°.

Therefore, the drop from the inside air to inside plywood surfaces will be $\frac{0.61}{5.40} \times 70^\circ$ or 7.90°: through the 1/4″ plywood, the drop is $\frac{0.32}{5.40} \times 70^\circ = 4.1^\circ$; and so on, until 0° temperature is reached at the outside air (see figure 2).

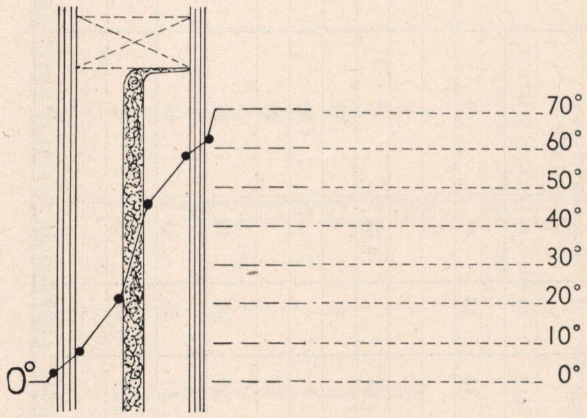


Fig. 2

Temperature Gradient through plywood wall.

TABLES OF INSULATION VALUES:

The tables on pages 4 to 8 give the resistances, and also the U values, for the component parts of typical plywood, and a few other, wall, roof, and floor constructions.

The resistance values (1/k) are taken from the A. S. of H. & V. E. guide or from latest Bureau of Standards tests. For Douglas fir plywood a resistivity of 1.28 is used; for batt insulation, 1/k=4.00; and for fibre board, 1/k=2.78 (see B.M.S. report No. 50, average of 17 commercial fibre boards).

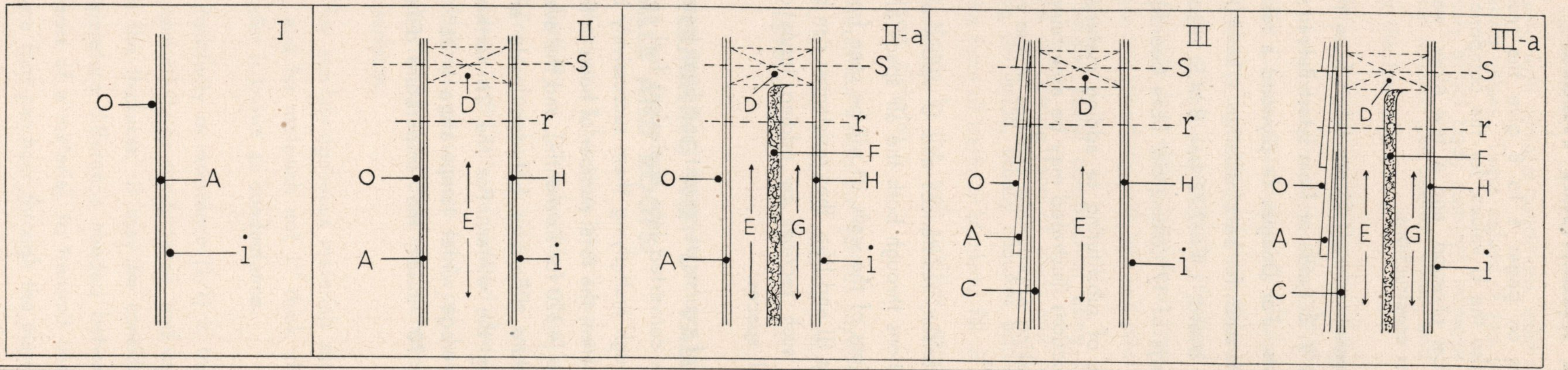
The effect of substituting or adding materials to the constructions illustrated may be ascertained quickly by making appropriate changes in the resistance values.

For comparison, resistances and U values are given for sections through both the air space and the stud portions of the walls. It will be seen from Table 1, Walls III and III-a, that resistances can be equalized at such sections by adding insulation within the wall spaces.

U values, of course, represent unit heat losses. These may be converted into equivalent fuel consumption through multiplying them successively by the total wall area, the total number of hours in the heating season (4,320 for six months), and the average temperature difference between inside and outside during the winter. For the 26 northern states, the average winter temperature is 36° F. If we assume 70° inside, the temperature difference is 34°.

"U" VALUES (COEFFICIENT OF HEAT TRANSMISSION) FOR WALLS EMPLOYING DOUGLAS FIR PLYWOOD

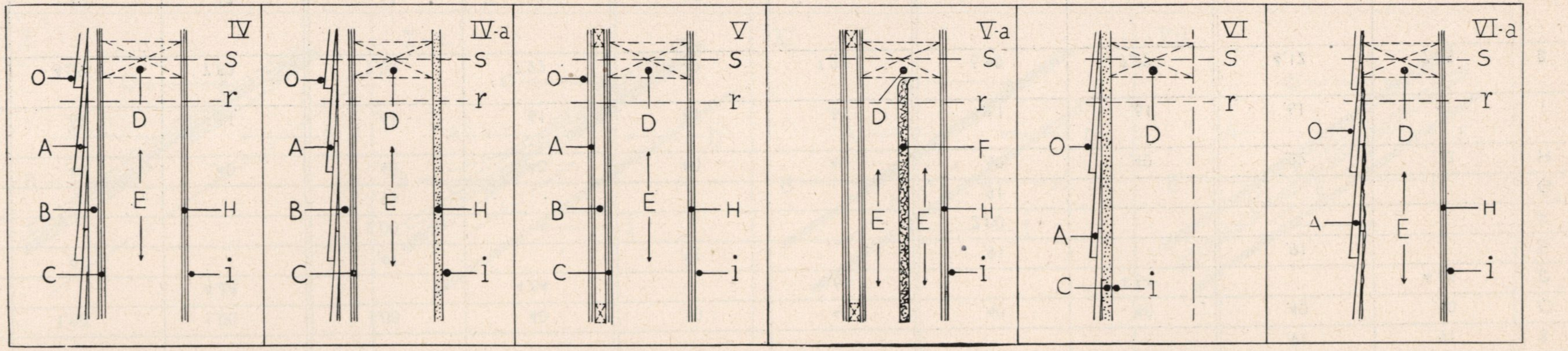
CROSS-SECTION of CONSTRUCTION



MATERIALS	SINGLE SHEET						DOUBLE WALL OF PLYWOOD						SHINGLES OVER PLYWOOD				
	Douglas Fir Plywood Thickness					Fibre Brd.	A—EXT. Plywood D—Stud E—Air Space H—INT. Plywood				Same as II but with ± 1" batt or blanket insulation A=⅜" and H=¼" plyw. & D=2⅝" stud		A—Wood Shingles and Bldg. Paper C—5/16" Plyscord H—¼" INT. Plywood		Same as III with Batt or Blanket Insulation		
	¼"	⅜"	½"	⅝"	¾"	½"	A—⅜" H—¼"		A—⅜" H—⅜"						At "r"	At "r"	
							At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	½" Insul.	1" Insul.	
RESISTANCES																	
Outside surface (15 mph wind) = O	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	O
Outside wall Covering A	.32	.48	.64	.80	.96	1.39	.48	.48	.48	.48	.48	.48	1.00	1.00	1.00	1.00	A
Furred Air Space B																	B
Sheathing C													.40	.40	.40	.40	C
Stud D								4.64		4.64		3.36		4.64			D
Air Space E							.91		.91		.91		.91		.91	.91	E
Insulation F											4.00				2.00	4.00	F
Air Space G											.91				.91	.91	G
Inside Wall Lining H							.32	.32	.48	.48	.32	.32	.32	.32	.32	.32	H
Inside Surface (still air) i	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	i
Total Resistance R	1.10	1.26	1.42	1.58	1.74	2.17	2.49	6.22	2.65	6.38	7.40	4.94	3.41	7.14	6.32	8.32	R
U=1/R (Btu's/hr/sq. ft/°temp.diff)	0.91	.79	.71	.63	.58	.46	.40	.16	.38	.16	.14	.20	.29	.14	.16	.12	U

"U" VALUES (COEFFICIENT OF HEAT TRANSMISSION) FOR WALLS EMPLOYING DOUGLAS FIR PLYWOOD

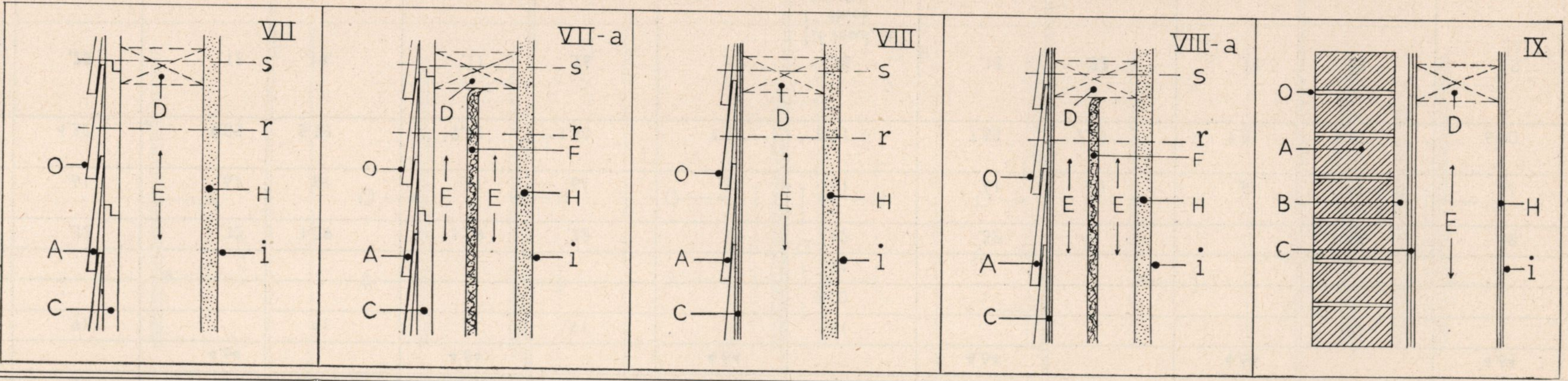
CROSS-SECTION of CONSTRUCTION



MATERIALS	DFP DRI-BILT CONSTR.		SAME AS IV		DFP DRI-BILT CONSTR. WITH PLYWOOD EXTERIOR		SAME AS V		FORMER C.C.C. CAMP CONSTR.		ALTERNATE FOR VI		
	A—Siding or Shingles B—Furred air space C—5/16" Plyscord D—3⅝" Stud H—¼" INT. Plywood		Except H—½" Fibre Brd. in place of ¼" Plywood Inside Wall		A—⅜" EXT. Plywood B—Furred air space C—5/16" Plyscord D—3⅝" Stud H—¼" INT. Plywood		Except ½" batt or blanket insulation is added at middle of air space		A—Siding and Paper C—½" Fibre Brd. No interior lining		A—Siding & Paper E—Air space H—⅜" INT. Plywood		
RESISTANCES	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	
Outside-surf. res. (15 mph wind) O	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	O
Ext. wall covering A	1.00	1.00	1.00	1.00	.48	.48	.48	.48	1.00	1.00	1.00	1.00	A
Furred air space B	.91	.91	.91	.91	.91	.91	.91	.91					B
Sheathing C	.40	.40	.40	.40	.40	.40	.40	.40	1.39	1.39			C
Stud D		4.64		4.64		4.64		4.64		4.64		4.64	D
Air space E	.91		.91		.91		.91				.91		E
Insulation F							2.00						F
Air space G							.91						G
Inside lining H	.32	.32	1.39	1.39	.32	.32	.32	.32			.48	.48	H
Inside surf. res. (still air) i	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	i
Total Resistance R	4.32	8.05	5.39	9.12	3.80	7.53	6.71	7.53	3.17	7.81	3.17	6.90	R
U=1/R (Btu's/hr/sq. ft/°temp.diff)	**.23**	.12	**.19.**	.11	**.26**	.13	**.15** (If insul. is 1" thick U=12)	.13	**.32**	.13	**.32**	.15	U

"U" VALUES (COEFFICIENT OF HEAT TRANSMISSION) FOR WALLS EMPLOYING DOUGLAS FIR PLYWOOD

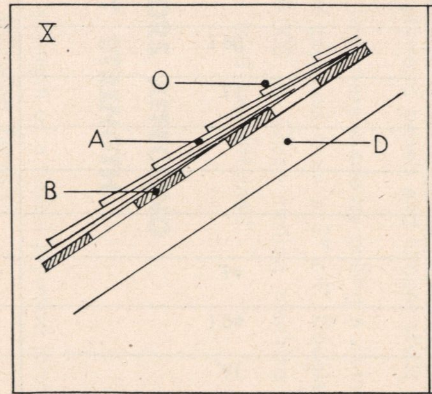
CROSS-SECTION
of
CONSTRUCTION

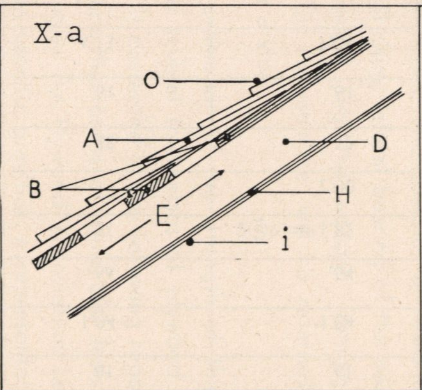


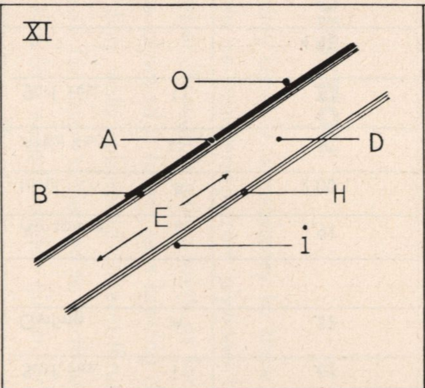
MATERIALS	OLD STYLE "STANDARD" CONSTR.		SAME AS VII		SIMILAR TO VII, BUT WITH 5/16" PLYSCORD SHEATHING		SAME AS VIII		BRICK VENEER		
	A—Siding or Shingles and Paper C—25/32" Shiplap D—3⅝" Stud H—¾" Wood Lath & Plaster		Except ½" batt or blanket insulation is added at middle of air space		A—Siding C—5/16" Plyscord D—3⅝" Stud H—¾" Wood Lath & Plaster		Except ½" batt or blanket insulation is added at middle of air space		A—4" Brick B—Air space C—5/16" Plyscord D—3⅝" Stud H—¼" INT. Plywood		
RESISTANCES	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	At "r"	At "s"	
Outside surf. Res. (15 mph wind) O	.17	.17	.17	.17	.17	.17	.17	.17	.17	.17	O
Ext. wall cov'g A	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	.80	.80	A
Furred air space B									.91	.91	B
Sheathing C	1.00	1.00	1.00	.40	.40	.40	.40	.40	.40	.40	C
Stud D		4.64		4.64		4.64		4.64		4.64	D
Air Space E	.91		.91		.91		.91		.91		E
Insul. F			2.00				2.00				F
Air Space G			.91				.91				G
Inside lining H	.40	.40	.40	.40	.40	.40	.40	.40	.32	.32	H
Inside surf. Res. (still air) i	.61	.61	.61	.61	.61	.61	.61	.61	.61	.61	i
Total Resistance R	4.09	7.82	7.00	7.22	3.49	7.22	6.40	7.22	4.12	7.85	R
U=1/R (Btu's/hr/sq. ft/°temp. diff)	.25	.13	.14	.14	.29	.14	.16	.14	.25	.13	U

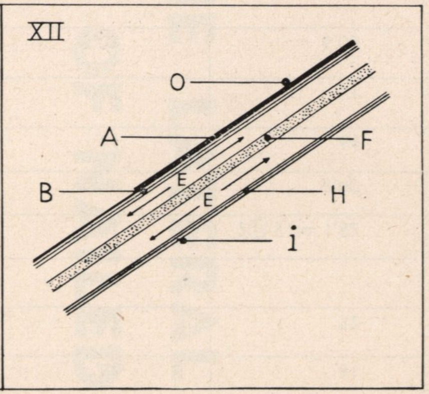
"U" VALUES (COEFFICIENT OF HEAT TRANSMISSION) FOR ROOFS

CROSS-SECTION of CONSTRUCTION





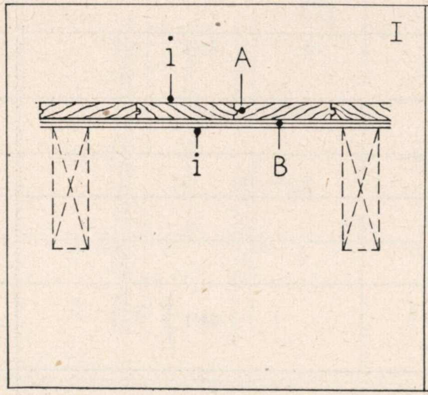


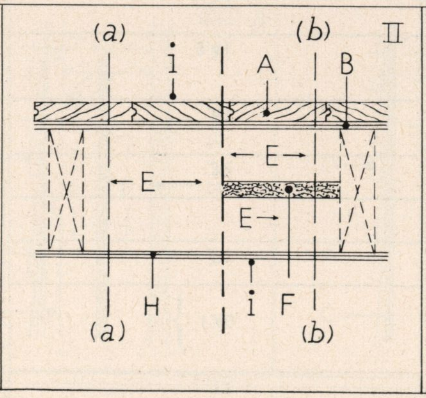


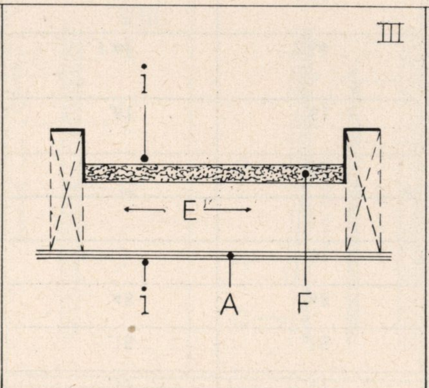
MATERIALS	WOOD SHINGLES ON STRIPS	SIMILAR TO X, BUT WITH PLYWOOD CEILING	COMPOSITION ROOFING OVER ⅜" PLYSCORD SHEATHING—WITH AND WITHOUT CEILING		PLYWOOD ROOF PANEL WITH COMPOS. ROOFING AND INSULATION	
	A—Shingles B—Strips (1"x4"), 2" apart D—Rafter	A—Wood Shingles B—5/16" Plyscord or 1"x4" Strips 2" apart H—¼" INT. Plywood	A—Comp. Roofing B—⅜" Plyscord D—Rafter H—¼" INT. Plywood (col. "b")		A—Comp. Roofing B—⅜" Plyscord E—Air spaces F—½" Batt or Blanket Insul. H—¼" INT. Plywood	
RESISTANCES			(a) No Ceiling Lining	(b) ¼" Plywood Ceiling		
Ext. surf. res. (15 mph wind) O	.17	.17	.17	.17	.17	O
Roofing A	1.40	1.40	.15	.15	.15	A
Sheathing B			.48	.48	.48	B
Air Space E		.91		.91	1.82 (Two air spaces @ .91)	E
Insulation F					2.00	F
Ceiling Lining H		.32		.32	.32	H
Inside surf. res. (still air) i	.61	.61	.61	.61	.61	i
Total Resistance R	2.18	3.41	1.41	2.64	5.55	R
U=1/R (Btu's/hr/sq. ft/°temp.diff)	0.46* (*from A.S.H. & V..E. guide)	.29	.71	0.38	.18	U

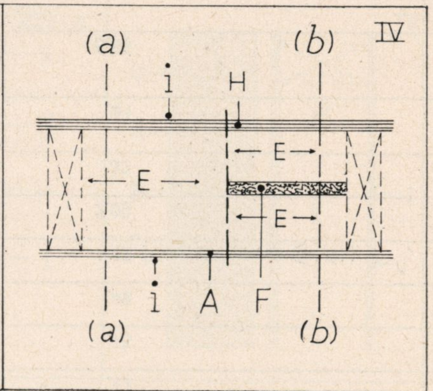
CEILINGS

CROSS-SECTION of CONSTRUCTION









MATERIALS		DOUGLAS FIR FINISH FLOOR OVER PLYWOOD SUB-FLOOR	SAME, WITH ¼" PLYWOOD CEILING		PLYWOOD CEILING WITH BATT OR BLANKET INSULATION			PLYWOOD ATTIC FLOOR AND PLYWOOD CEILING		
		A—13/16" D. Fir Floor B—½" Plyscord Sub-floor	A—13/16" D. Fir Floor B—½" Plyscord Sub-floor E—Air spaces F—½" batt or blanket insulation H—¼" INT. Plywood		A—¼" INT. Plywood F—½" batt or blanket			A—¼" INT. Plywood F—½" Batt or Blanket H—½" Plyscord		
RESISTANCES			At (a) (No insul.)	At (b) (½" batt or blanket)				At "a-a" (No insul.)	At "b-b" (½" batt or blanket)	
Surf. resist. (room)	i	.61	.61	.61	Surf. res.	i	.61	.61	.61	i
Finish flooring	A	1.04	1.04	1.04	Ceiling	A	.32	.32	.32	A
Sub-floor	B	.64	.64	.64						B
Air space	E		.91	2@91= 1.82	Air space	E	.91	.91	2@.91= 1.82	E
Insulation	F			2.00	Insul.	F	2.00		2.00	F
Ceiling	H		.32	.32	Attic floor	H		.64	.64	H
Surf. Resist. (still air)	i	.61	.61	.61	Surf. res.	i	.61	.61	.61	i
Total Resistance	R	2.90	4.13	7.04			4.45	3.09	6.00	R
U=1/R (Btu's/hr/sq. ft/° temp.diff)		.35	.24	.14			.23	.32	.17	U

SECTION SIX

Revised November 1, 1948

THE LATERAL BEARING STRENGTH OF NAILED PLYWOOD JOINTS

INTRODUCTION

In many cases it is necessary to know the loads which can be transferred from plywood to a framing member through a nailed joint, as for example, when the plywood is to act as a bracing diaphragm for building walls subjected to end-thrust due to wind loads, for floors subjected to earthquake loads, for the web of a nailed wooden girder, or as gusset plates in trusses. The direction of stress in all these cases will cause lateral bearing of the nail on both the plywood and the wooden framing member which, for convenience, will be referred to hereafter as a stud.

An evaluation of the strength of this type of joint for several thicknesses of plywood and sizes of nails is presented in the following report. These values are the results of some 650 tests made in the Douglas Fir Plywood Association Laboratory. Based on specific gravities, a representative selection of plywood panels and lumber studs was used.

The tests were made by pulling a six-inch square piece of plywood from a 2″ x 4″ Douglas fir stud in a 10,000 lb. Olsen testing machine until the ultimate load was attained.

FACTORS AFFECTING JOINT STRENGTH

The strength of a nailed plywood joint depends on the following factors:

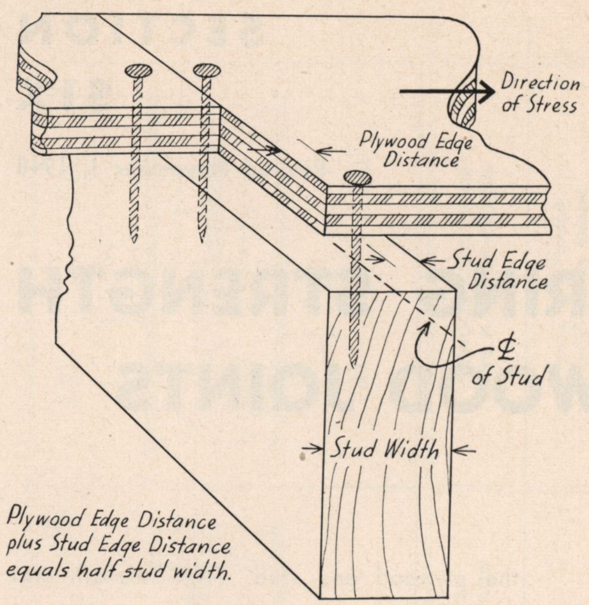
1. The thickness of plywood.
2. The size of the nail.
3. The moisture content and specific gravity of the plywood and stud. (The strength also depends upon the species of the stud.)
4. The edge-distance of the plywood, i.e., the distance from the center of the nail to the edge of the plywood in the direction of stress.
5. The edge-distance of the stud.

(Although it was suspected that the strength of the joint would depend upon the direction of the face-ply grain of the plywood with respect to the load, careful tests on joints loaded parallel, perpendicular, and at 45° to the face-ply grain showed no consistent or predictable difference. Furthermore, the direction of load with respect to the grain of the stud is not a factor, provided, of course, that end-grain nailing is not used.)

The plywood thicknesses tested were 5/16″, 3/8″, 1/2″, 5/8″ and 3/4″. The nail sizes tested were 6d com., 8d com., 10d com. and 16d com. The specific gravities of the specimens, both plywood and studs, were those ordinarily encountered for Douglas fir. The moisture contents varied from 11% to 20% for the studs and from 8% to 10% for the plywood. The edge-distances were varied from 1/4″ to 1″.

METHOD OF ANALYSIS

It is evident that to develop the maximum strength of a nailed joint, it is necessary to have a certain minimum edge-distance for both the plywood and the stud. Accordingly two series of tests were conducted to ascertain these minimum edge-distances, which will be referred to as "Standard" edge-distances.



Plywood Edge Distance plus Stud Edge Distance equals half stud width.

Fig. 1

In the first series, the distance of the nail from the panel edge was varied, starting with ¼″ and increasing until it was evident that no further increase in load would occur. In all these tests the nail was driven at least 1″ from the stud edge.

These tests showed that as the edge-distance was decreased from the standard, the strength of the joint also decreased along definite curves, depending upon the thickness of the plywood, as shown in Figure 2.

In these curves, plotted on a basis of percentages, the standard edge-distance is taken as 100%, and the ultimate load at this edge-distance, as shown in the Table below Figure 2, is also taken as 100%. The curves show the effect on the ultimate load when the nail is driven into plywood at varying edge-distances. Accordingly Figure 2, in conjunction with the Table below it, permits a ready determination of the ultimate nail bearing load for any of the combinations of nails, edge-distances, and plywood thicknesses. (See example on Page 4.)

In the second series, the procedure was reversed, varying the stud edge-distance from ¼″ to an optimum distance, but maintaining a plywood edge-distance of at least 1″. The standard stud edge-distance was found to be 1″ for the 6d, 8d and 10d nails, and ⅞″ for the 16d nails. The maximum nail bearing values developed for the studs are naturally equal to the maximum developed for the same size nail in the plywood, although the corresponding standard edge-distances differ somewhat.

The effect of using stud edge-distances less than the standard is shown by curves in Figure 3, for the four sizes of nails tested. The plywood thicknesses for the panels tested had little effect on these values and is not a factor here, although it would be for plywood so thick that the nails could not penetrate into the stud a distance equal at least to two-thirds of their shank length.

In the majority of cases, when plywood is applied to framing members, the available edge-distances are less than the standard values given in the Table below Figure 2 for both plywood and stud; consequently the ultimate loads will be reduced. An illustration of such reduced edge-distances is given in Figure 1, where two panels of plywood are butt-jointed over a 1⅝″ stud. Thus a ⅜″ plywood edge-distance and a 7/16″ stud edge-distance may be used.

The ultimate strength of such joints will be limited by the nail-bearing value of either the plywood or the stud.

The plywood bearing value may be obtained from Figure 2.

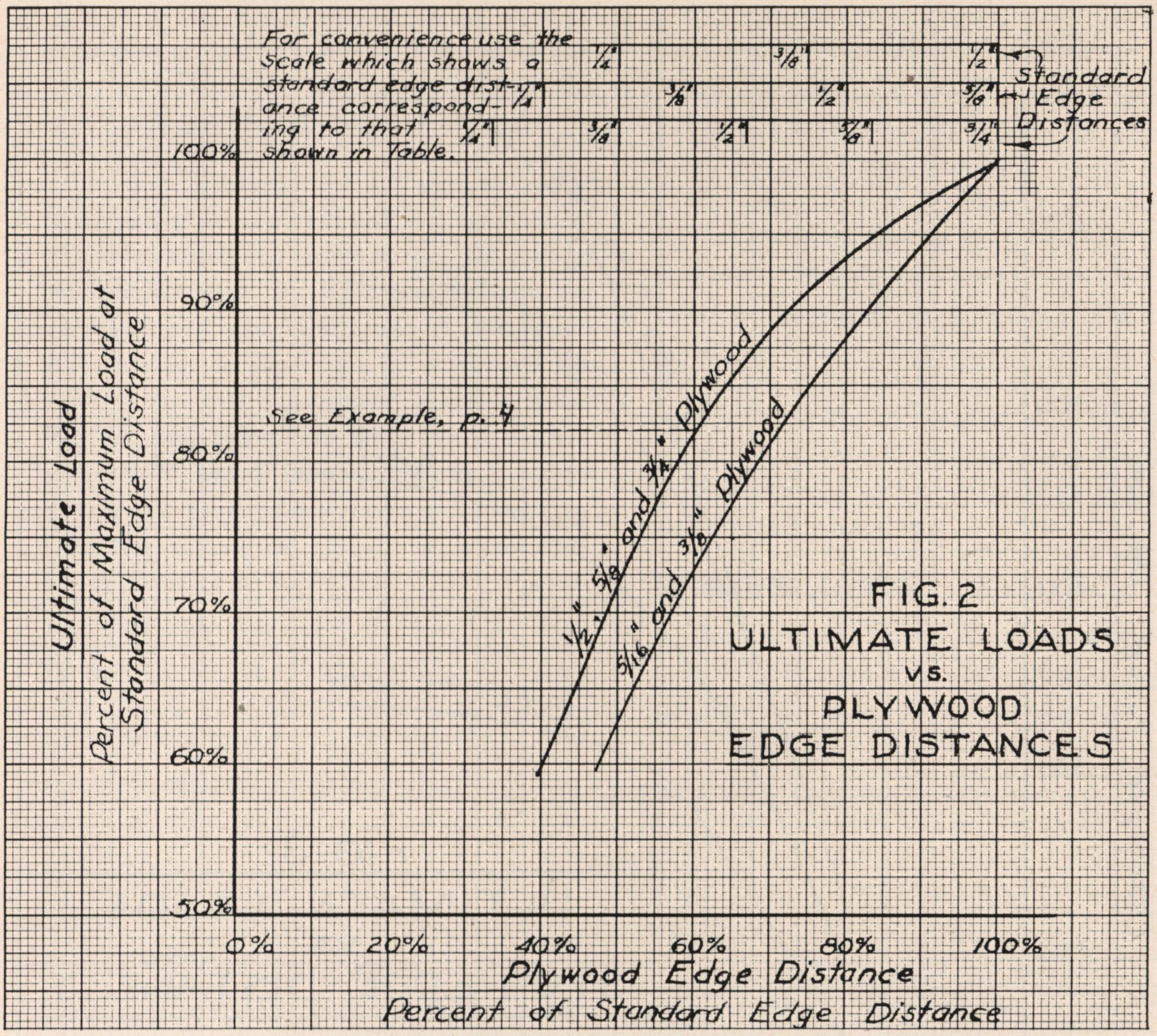


FIG. 2
ULTIMATE LOADS
VS.
PLYWOOD
EDGE DISTANCES

PLYWOOD THICKNESSES	NAIL SIZE							
	6d COM.		8d COM.		10d COM.		16d COM.	
	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE
5/16"	335#	1/2"	420#	5/8"				
3/8"	345#	1/2"	470#	5/8"				
1/2"			470#	5/8"	500#	1/2"		
5/8"			470#	5/8"	575#	5/8"	620#	5/8"
3/4"					575#	3/4"	675#	3/4"

Ultimate Loads at Standard Plywood Edge-Distances for Laterally Loaded Nail Joints.

(For stud edge-distances one inch or greater.)

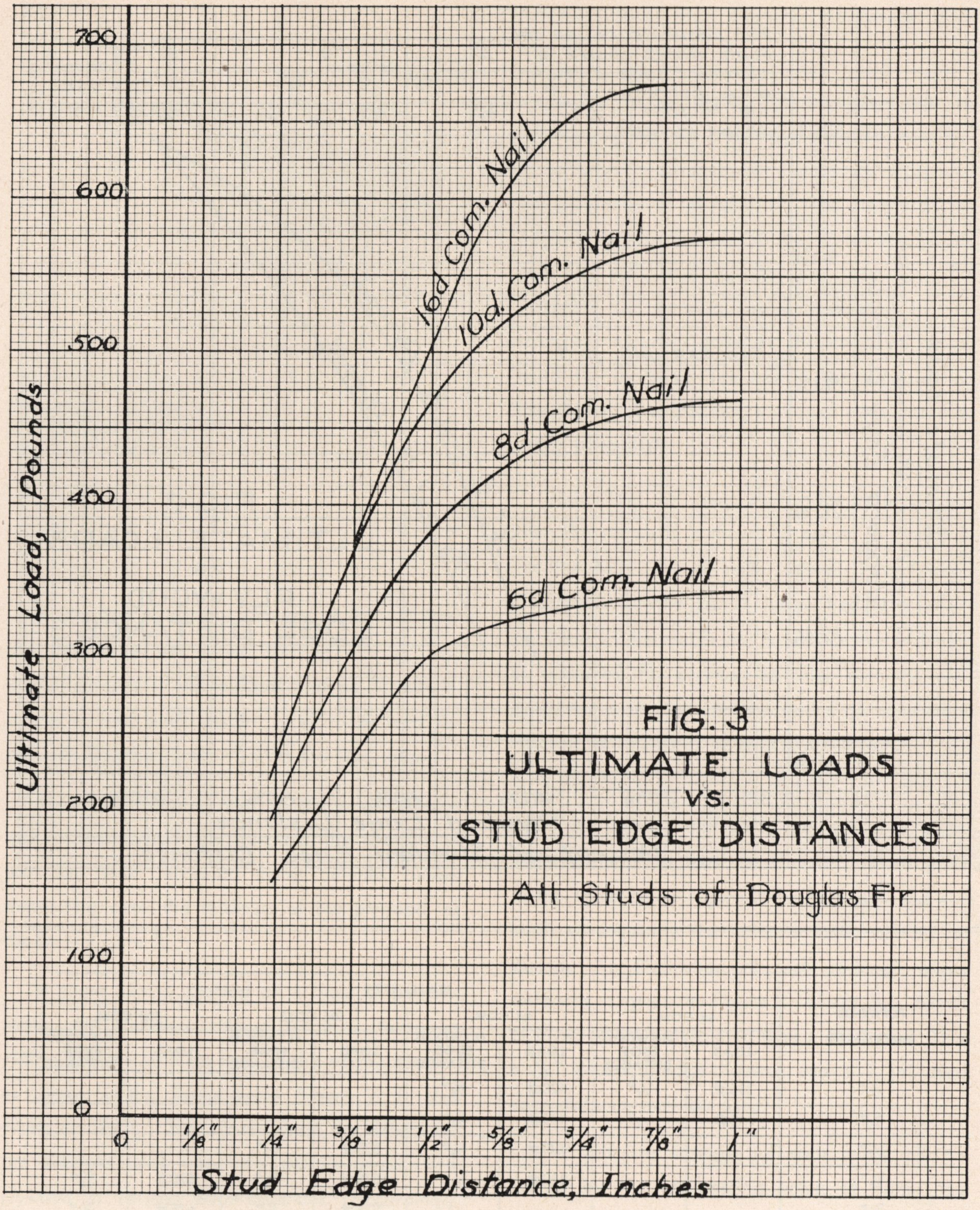


FIG. 3
ULTIMATE LOADS
VS.
STUD EDGE DISTANCES
All Studs of Douglas Fir

EXAMPLE:

First, select from the Table the ultimate load value at **standard** edge-distance for the plywood thickness and nail size contemplated. For 1/2" plywood butted at the center line of a 1 5/8" stud and 8d com. nails, the value is 470 lbs. at 5/8" edge-distance. If nails are to be driven 3/8" from plywood edge, the edge-distance is, of course, 60% of the standard. Using this value for the abscissa in Figure 2, as shown by the dotted lines, the ultimate bearing value is seen to be 82% of that for the 5/8" standard, or 82% of 470, which is 385 lbs. per nail.

This value must be checked against the bearing value of an 8d com. nail in the stud. From Figure 3, it is seen that an 8d nail, with 7/16" edge-distance (which corresponds with the ⅜″ plywood edge-distance as shown in Fig. 1), has a bearing value of 349 lbs. This will govern since it is less than the plywood bearing value of 385 lbs.

It will be noted that the ⅜″ edge-distance in the plywood was selected arbitrarily, and that it is not necessarily the distance that will develop the optimum value. For more precise design, the optimum value may be obtained by a cut and try procedure. A slight increase in the stud distance to ½″ and a consequent decrease in the plywood edge-distance to 5/16″ would mean a value of 385 lbs. for the stud but only 71½% of 470, or 335 lbs. for the plywood, which would be lower than before. The optimum theoretical value will be found to be 365 lbs. at 11/32″ edge-distance in the plywood and 15/32″ (i.e., $\frac{1\frac{5}{8}''}{2} - \frac{11''}{32}$) for the stud. Obviously some tolerance for placing of nails must be provided for in design, and no attempt has been made to carry values in Tables I and II closer than 1/16th of an inch.

For the convenience of the designer, Tables I and II have been computed to show the ultimate loads of plywood joints butted on the centerline of 1⅝″ and 2⅝″ studs. (Douglas fir lumber studs 3½″ or thicker will develop the full strength of all combinations of nails and plywood shown when the plywood is butted on the center-line of the stud.) In each case edge-distances have been chosen so that maximum joint strength will result; any deviation from these edge-distances will result in weaker joints. These tables should be used, of course, only in conjunction with a suitable factor of safety. (See following paragraph.)

WORKING LOADS

The translation of the actual ultimate loads given in Tables I and II, to working loads that may be safely used under given conditions should be

TABLE I.

PLYWOOD THICKNESSES	NAIL SIZE							
	6d COM.		8d COM.		10d COM.		16d COM.	
	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE
5/16″	275#	⅜″	305#	⅜″				
⅜″	275#	⅜″	340#	⅜″				
½″			350#	⅜″	425#	⅜″		
⅝″			350#	⅜″	425#	⅜″	445#	⅜″
¾″					410#	⅜″	445#	⅜″

Maximum Ultimate Loads of Plywood Nailed to 1⅝″ Douglas Fir Lumber

Plywood panels butted at the center lines of studs.
(Any change in edge-distance will decrease load.)

TABLE II.

PLYWOOD THICKNESSES	NAIL SIZE							
	6d COM.		8d COM.		10d COM.		16d COM.	
	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE	LOAD	PLYWOOD EDGE-DISTANCE
5/16″	335#	½″	420#	⅝″				
⅜″	340#	½″	445#	9/16″				
½″			455#	9/16″	500#	⅝″		
⅝″			455#	9/16″	555#	9/16″	620#	⅝″
¾″					540#	⅝″	640#	⅝″

Maximum Ultimate Loads of Plywood Nailed to 2⅝″ Douglas Fir Lumber

Plywood panels butted at the center lines of studs.
(Any change in edge-distance will decrease load.)

largely a matter of judgment for the individual designer. However, it may be of assistance to point out some of the factors which should be considered.

As nailed joints are loaded they deform at a varying rate, as illustrated in Figure 5. It can be seen that there is no true proportional limit, but rather a region in which the rate of deformation with respect to the load increases markedly. This region occurs at a load of approximately 25% or 30% of the ultimate and will, for convenience, be called the proportional limit. In no case should working loads exceed the proportional limit as excessive deformation is likely to result.

The nailing properties of Douglas fir are subject to a variation of approximately 25% both above and below the average, as illustrated by typical curves found for ultimate load vs. stud edge-distance, Figure 4. (The extreme spread of values illustrates the use of exceptionally light and heavy studs.) Thus for conservative design, it may be desirable to reduce the average ultimate loads by 25%.

The effect of seasoning on nailed joints is to reduce their ultimate strength by varying amounts. The reduction is too uncertain to permit accurate evaluation; however, a series of tests showed a loss in ultimate strength of 25% for seasoned specimens over control specimens tested immediately after nailing. The seasoning consisted of placing the nailed specimens in a humidity chamber to increase their moisture content, removing to dry out, replacing in the chamber, and testing upon removal. The average change in moisture content was 5% for the studs and 9% for the plywood, conditions which might readily occur in ordinary sheltered construction. The loss in strength is probably just as severe when the stud is green and seasons under load, but a 25% reduction for seasoning is probably adequate.

The length of time during which load is applied to the nailed joint may affect its ultimate load capacity in the same manner that it affects the strength of a wooden beam. Such a beam, for example, when loaded for several years, will rupture under a load equal to only 9/16th of that load which would cause failure when applied for the few moments required by the usual test. Therefore, a factor of 9/16th may be applied to the ultimate nail loads to allow for long time loading.

Other conditions that may cause a reduction in strength are repetition of load and reversal of stress. To provide for these factors, and as a guard against overloading, it may be well to add a factor of safety.

In choosing the working load for a nailed joint, the designer may wish to allow for some or all of the above-mentioned reductions by the simple application of a numerical factor. For instance, the division of the ultimate loads found in this series of tests by a factor of slightly less than five gives values which are in close agreement with working loads for Douglas fir plywood recommended in the Uniform Building Code adopted by Pacific Coast Building Officials Conference.

The same factor for determining working loads in joints nailed at standard edge-distance should be applied also to joints having reduced edge-distances.

It is believed that values can be interpolated for other nail sizes by making use of data developed by the U. S. Forest Products Laboratory, indicating that the lateral bearing strength of nails is proportional to the 3/2 power of the nail diameter. The values may also be applied, with judgment, when the stud material is wood other than Douglas fir, by a comparison of the species of the wood to be used with that of Douglas fir, as illustrated on Page 123 of the Forest Products Laboratory's "Wood Handbook."

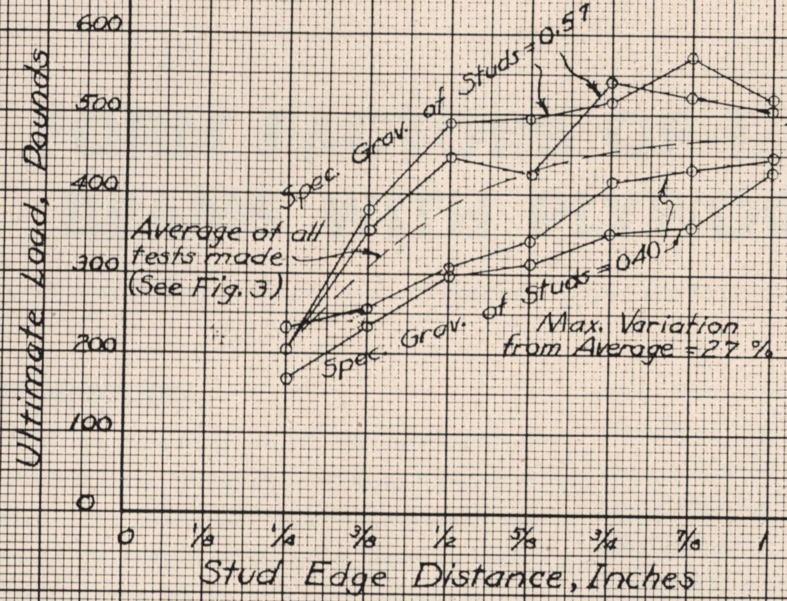


FIG. 4

TYPICAL CURVES

ULTIMATE LOAD vs. STUD EDGE DISTANCE

½" Plywood
Douglas Fir Studs
8d. Com. Nail

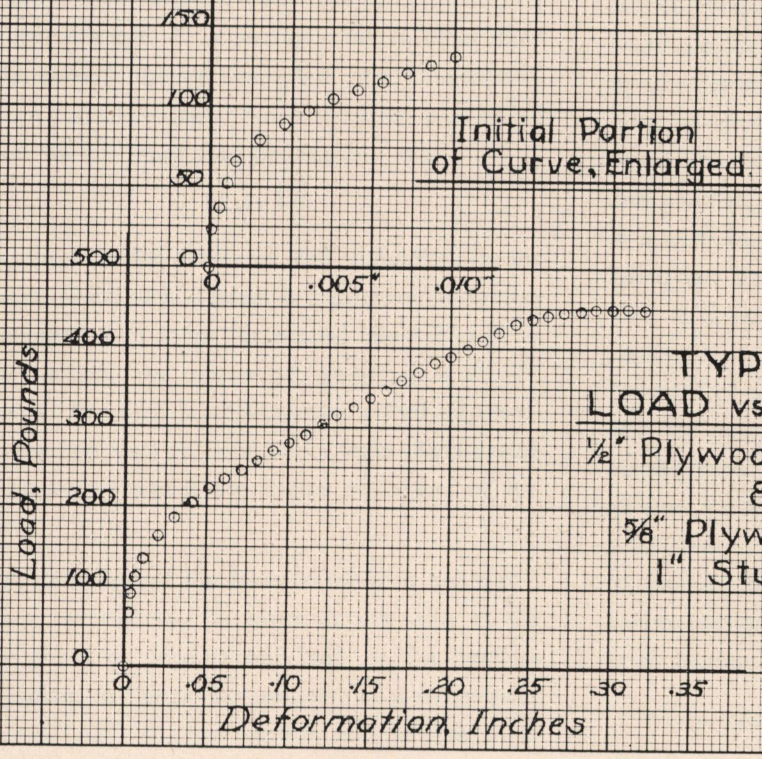


FIG. 5

TYPICAL CURVE

LOAD vs. DEFORMATION

½" Plywood. Douglas Fir Stud.
8d Com. Nail.
⅝" Plywood Edge Distance
1" Stud Edge Distance

SECTION SEVEN

Revised November 1, 1948

FORM FACTORS OF I AND BOX BEAMS WITH PLYWOOD WEBS

(The following section is based upon data published by the United States Forest Products Laboratory, Madison, Wisconsin.)

INTRODUCTION

With the increasing use of built-up beams of I and box sections having plywood webs it becomes necessary to understand more completely the nature of the stresses produced in them by bending loads in order to design a safe, yet economical section.

The usual engineering formulae can be applied in calculating the safe load of a given built-up section, providing they are properly modified by certain known factors. For example, when the flexural strength of a section that is other than rectangular is being computed, the conventional flexure formula is used, but in addition a so-called "form factor" must also be applied; thus

$$M = \frac{FsI}{c}$$

This form factor indicates the reduction in the stresses which such a beam is capable of sustaining as compared to those for a solid beam 2″ x 2″ in cross section. The reason for specifying a 2″ x 2″ beam as the basis for comparison is that all standard bending tests on clear wood, from which design values are obtained, are based on beams of this type.

A definition of the form factor of a beam of I or box section may be given as the ratio of the fiber stress at proportional limit, or the modulus of rupture, to the similar property of a rectangular beam 2″ x 2″ in cross section and of the same material as in the flanges of the I or box beam.

REASON FOR FORM FACTOR

The reason why built-up beams reach their proportional limits and ultimate loads at stresses lower than corresponding 2″ x 2″ sections may be understood by considering the nature of wood when subjected to a bending moment. It is well known that the strength of wood in tension is far greater than in compression; thus, when a wood beam fails, it first gives way on the compression face. Furthermore, it has been observed that the computed fiber stress at the proportional limit in bending is far greater than the fiber stress at proportional limit in compression parallel to the grain, where the stress is distributed uniformly across the grain. The ability to take a greater stress when subjected to bending may be accounted for by considering the wood fibers to act as miniature Euler columns, more or less bound together. These fibers when all stressed alike offer little support to one another, but when the stress is non-uniform as in a bent beam, the fibers nearer the neutral axis, being less stressed, will not buckle, and will therefore lend lateral support to the extreme fibers, causing them to take a higher load. However, in the case of an I or box beam, some of the fibers near the neutral axis have been removed, with the consequence that less supporting action is available to the extreme fibers than if the beam were solid. Thus it is necessary to apply a form factor to built-up beams in order to use the ordinary lumber design values which are based on tests of small beams.

(For a more detailed explanation of the internal stresses in a wooden beam and the derivation of the form factor formulae, reference should be made to the United States Forest Products Laboratory Bulletin No. 1310, "Form Factors of Beams Subjected to Transverse Loading Only" and A. S. T. M.

Bulletin No. 118, October, 1942, "Stress-Strain Relations in Timber Beams of Douglas Fir" by Professor Albert Dietz, Massachusetts Institute of Technology.)

FORMULAE

Equations which evaluate the form factor of all ordinary beam sections have been determined by the United States Forest Products Laboratory and are presented in the Wood Handbook published by that institution. That one which is of principal importance in the design of built-up plywood beams has to do with the form factor at proportional limit of I and box beams, and is expressed as follows:

$$F_{p.l} = 0.58 + 0.42 \left. \frac{K(t_2 - t_1)}{t_2} + \frac{t_1}{t_2} \right\},$$

where K is a value, determined by experiment, which depends on the ratio of the depth of the compression flange to the depth of beam. This relationship is shown in the following table:

Ratio of depth of compression flange to depth of beam d/h	Values of K
0.10	0.085
0.15	.155
0.20	.230
0.25	.315
0.30	.400
0.35	.490
0.40	.575
0.45	.660
0.50	.740
0.55	.810
0.60	.875
0.65	.920
0.70	.950
0.75	.970
0.80	.985
0.85	.995
0.90	.998
0.95	1.000
1.00	1.000

The total width of beam is t_2, and t_1 is the total width of webs. For convenience in design a graphical solution of this formula is presented on a following page.

The form factor for modulus of rupture is slightly lower than for proportional limit, and is given by the equation

$$F_r = 0.50 + 0.50 \left\{ \frac{K(t_2 - t_1)}{t_2} + \frac{t_1}{t_2} \right\},$$

where the symbols have the same meaning as above. This formula need only be applied when checking the ultimate load on a built-up beam. By combining the two formulae it can be seen that

$$F_r = 1.19 \, F_{p.l.} - 0.19$$

As stated above, values of $F_{p.l.}$ can be conveniently determined from the accompanying chart.

DEPTH FACTOR

The form factor discussed in the previous section should not be confused with the **depth** factor, which is the ratio of the unit strength value of a **rectangular** beam to the corresponding unit value for a beam 2″ x 2″ in cross section. It is known that as the height of a rectangular beam is increased the modulus of rupture decreases.

The depth factor may be calculated by the formula

$$F_d = 1.00 - 0.07 \left\{ \sqrt{d/2} - 1 \right\},$$

where d is the depth of the beam in inches.

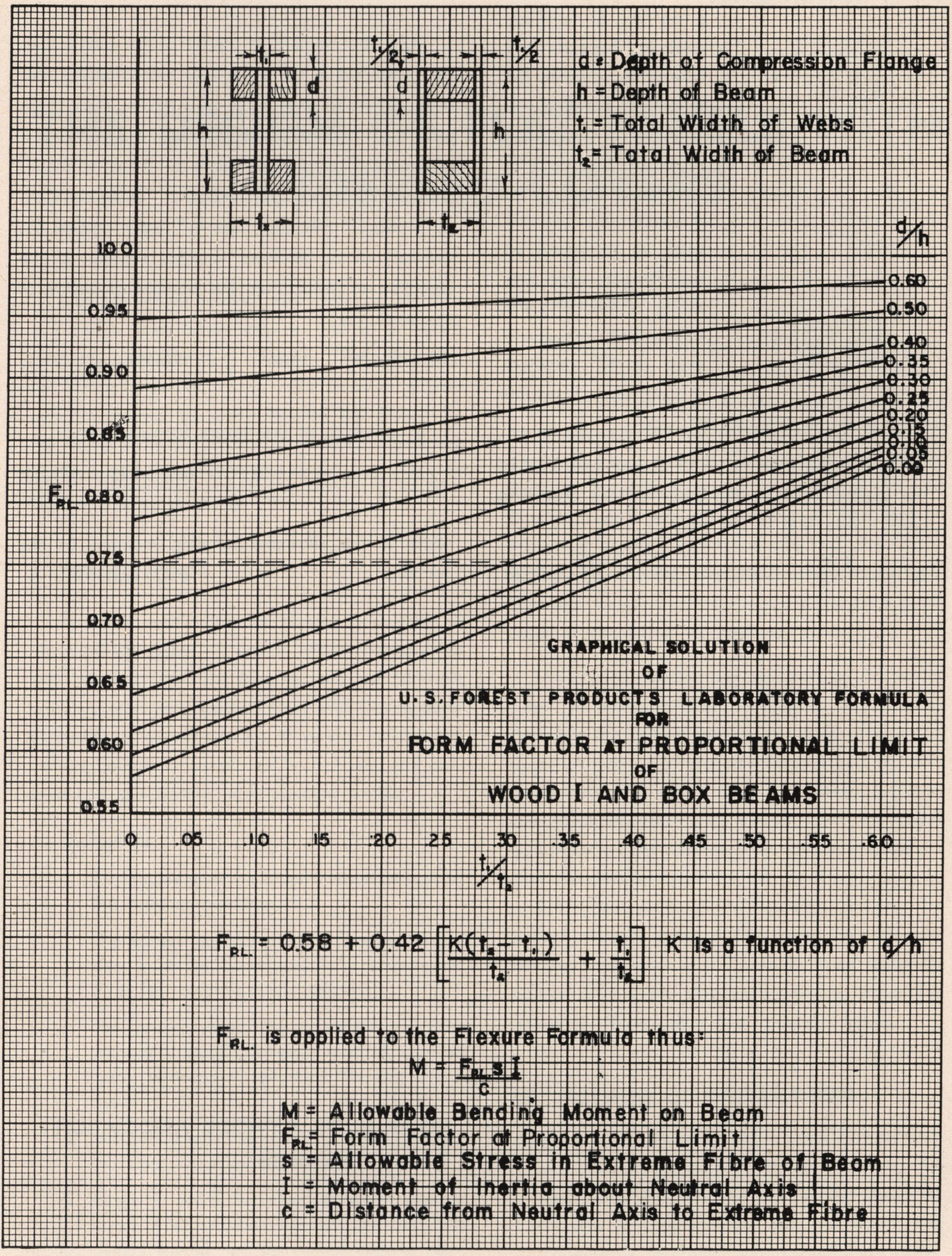
When making accurate comparisons of rectangular beams of different depths, this depth factor should be applied to the flexure formula. For example, the depth factor of a beam 42″ deep is 0.75. This suggests that if a rectangular beam is fairly deep—say, over 16″—its true depth factor should be applied in the flexure formula. However, it need not be considered in the usual design of joists and girders, because when working stresses for structural lumber were set up there was included an average depth factor of 0.90 which corresponds to a 12″ depth of beam.

An I or box beam need not have a depth factor applied in addition to the form factor because in the derivation of the expression for form factor, the depth of beam was one of the variables considered.

Moreover, when a form factor is applied to an I or box beam, it should be remembered that a depth factor of 0.90 has already been applied to the working stress of the lumber in the flanges, and consequently the working stress may be multiplied by 10/9ths before applying the accurately calculated form factor.*

---

*An increase of 20% in the working stresses for lumber was authorized August 9, 1943 by the War Production Board, to apply for the duration of the war to all government-controlled construction. For this reason application of the 10/9ths factor to these increased working stresses may be optional with the designer.



d = Depth of Compression Flange
h = Depth of Beam
t_1 = Total Width of Webs
t_2 = Total Width of Beam

GRAPHICAL SOLUTION
OF
U. S. FOREST PRODUCTS LABORATORY FORMULA
FOR
FORM FACTOR AT PROPORTIONAL LIMIT
OF
WOOD I AND BOX BEAMS

$$F_{P.L.} = 0.58 + 0.42 \left[\frac{K(t_2 - t_1)}{t_2} + \frac{t_1}{t_2} \right]$$ K is a function of d/h

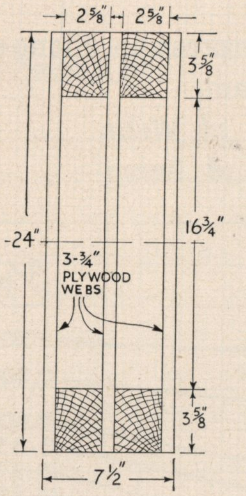
$F_{P.L.}$ is applied to the Flexure Formula thus:

$$M = \frac{F_{P.L.}\, s\, I}{c}$$

M = Allowable Bending Moment on Beam
$F_{P.L.}$ = Form Factor at Proportional Limit
s = Allowable Stress in Extreme Fibre of Beam
I = Moment of Inertia about Neutral Axis
c = Distance from Neutral Axis to Extreme Fibre

NUMERICAL EXAMPLE

An illustration of the application of the form factor to built-up plywood beams can best be given by a numerical example. Suppose it is desired to compute the amount of bending moment which can be resisted by the multiple-web section shown in the accompanying sketch, which is composed of two flanges, each consisting of two 3″ x 4″s, sandwiched between 3 webs of ¾″ thickness. Assume an allowable stress in the lumber of 1600 p.s.i.



To find the form factor at proportional limit of the section, refer to the graph on the preceding page. Note that in this case

$$t_1/t_2 = \frac{3(.75)}{7.5} = 0.300 \text{ and } d/h = \frac{3.625}{24} = 0.151$$

The dotted line on the graph shows that the corresponding $F_{\text{p.l.}}$ is 0.752. This form factor can then be increased by 10/9ths as explained above.

Next the moment of inertia of the section can be computed in the usual way, remembering that in the plywood only those plies having their grain parallel to the direction of the stress are considered in **flexure.** (This restriction does not apply to shear.) Reference to page 10, section 2, "Technical Data on Plywood" discloses that ¾″ 5-ply sanded has one-half of its thickness running parallel to the direction of stress. Then the moment of inertia of the section is

$$\frac{2(2.625)}{12}(24^3 - 16.75^3) + \frac{3(.75)(24^3)}{2(12)} = 5{,}285 \text{ in.}^4$$

Application of the flexure formula gives

$$M = \frac{10}{9}\,\frac{F_{\text{p.l.}}sI}{c} = \frac{(10)(.752)(1600)(5{,}285)}{(9)\qquad 12} = 589{,}000 \text{ in. lbs.}$$

which is the allowable bending moment on the section.

DOUGLAS FIR PLYWOOD ASSOCIATION
Tacoma 2, Washington

SECTION
EIGHT

Revised November 1, 1948

THE DESIGN OF FLAT PANELS WITH STRESSED COVERS

Comprehensive studies and tests on stressed skin panels have been made by the U. S. Forest Products Laboratory. A bulletin, "The Designing for Strength of Flat Panels with Stressed Coverings," prepared by John Newlin, was issued by the Forest Products Laboratory in 1940. The design methods presented in that bulletin are given in somewhat greater detail.

I. DEFINITION

A panel with stressed covers or simply a "stressed-skin panel" is one in which the covering acts integrally with the framing members to resist bending loads in proportion to its effective moment of inertia. It usually consists of several longitudinal framing members, separated by headers, and covered top and bottom with plywood panels.

A. Limitations.

1. The covers must be glued to both edges of the framing members.
2. The covers and longitudinal framing members must be continuous in a longitudinal direction, or adequately spliced.
3. The covers should be adequate to resist secondary bending stresses from anticipated loads between longitudinal framing members.
4. Headers should be provided with relatively thin deep longitudinal members.
5. Panels in which the clear distance between longitudinal members exceeds 2b* for both covers should not be considered to have a stressed skin.

II. DESIGN

Stressed skin panels may be designed in accordance with accepted engineering formulas, slightly modified because the proportions of the panel are such that the covers may tend to buckle under excessive loading. Therefore as the spacing between framing members is increased, the allowable load is reduced in two ways; first, less of the cover is considered to be effective when computing the moment of inertia and second, the allowable bending stresses are reduced to a minimum of two-thirds the ordinary value.

As ordinary beams are designed for horizontal shear, so is the stressed skin panel. In this case, however, the critical section usually occurs at the joints between the framing members and the plywood cover (or within the cover itself), because the stress involved is a rolling shear for which design values are relatively low. Thus it is apparent that the allowable load on the panel may be governed by the width of the framing members. The horizontal shear stress in the latter may also be computed if there is a possibility that seasoning checks will, in the designer's opinion, lower the allowable stress below the ordinary values.

III. METHOD OF CALCULATON

A. Assume the cross-section of panel to be checked. The design may be based either on the entire width of panel or an I section consisting of one framing member and its contributing cover area. Although the procedure is similar in both cases, it is simpler and ordinarily sufficiently accurate to use the latter method, which will be described in the following paragraphs.

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*See Paragraph III-B.

B. Determine the basic spacing "b" for top and bottom covers from Table 2, (page 4), using appropriate grain direction.

C. Locate the neutral axis of the I section by taking moments about some convenient point. Consider the cover width as extending half way to the adjacent framing members, unless the clear distance between members exceeds "b," in which case take the cover width equal to "b" plus the framing member thickness. Also consider only those plies in the covers whose grain is parallel to the span.

D. Find the moment of inertia of the section, considering the material used in locating the neutral axis.

E. Determine the allowable resisting moment of the section on both tension and compression faces, by means of the flexure formula. In each case use the appropriate distance from the neutral axis to the extreme fiber as determined in paragraph C. Use the appropriate stress as found in paragraph F, applying the allowable bending stress in the extreme fiber to the tension face, and the allowable stress in compression parallel to the compression face. The allowable bending moment on the panel will equal the smaller value of the resisting moment, in tension or compression as the case may be.

F. Working stresses for selected grades of Exterior Type Douglas fir plywood are given in Table I. (Working stresses for certain grades of Interior Type plywood differ somewhat, as shown in Section 2, Table I.) These are suitable for use with stressed skin panels, **provided** the clear distance between framing members is not greater than b/2.

When this spacing exceeds b/2, the recommended working stresses in both bending and compression parallel (but not in rolling shear) must be reduced proportionately from 100% at b/2 to 67% when the clear spacing is b or greater, as illustrated in Fig. I.

This reduction, as mentioned in Paragraph II, is to provide against buckling of the covers.

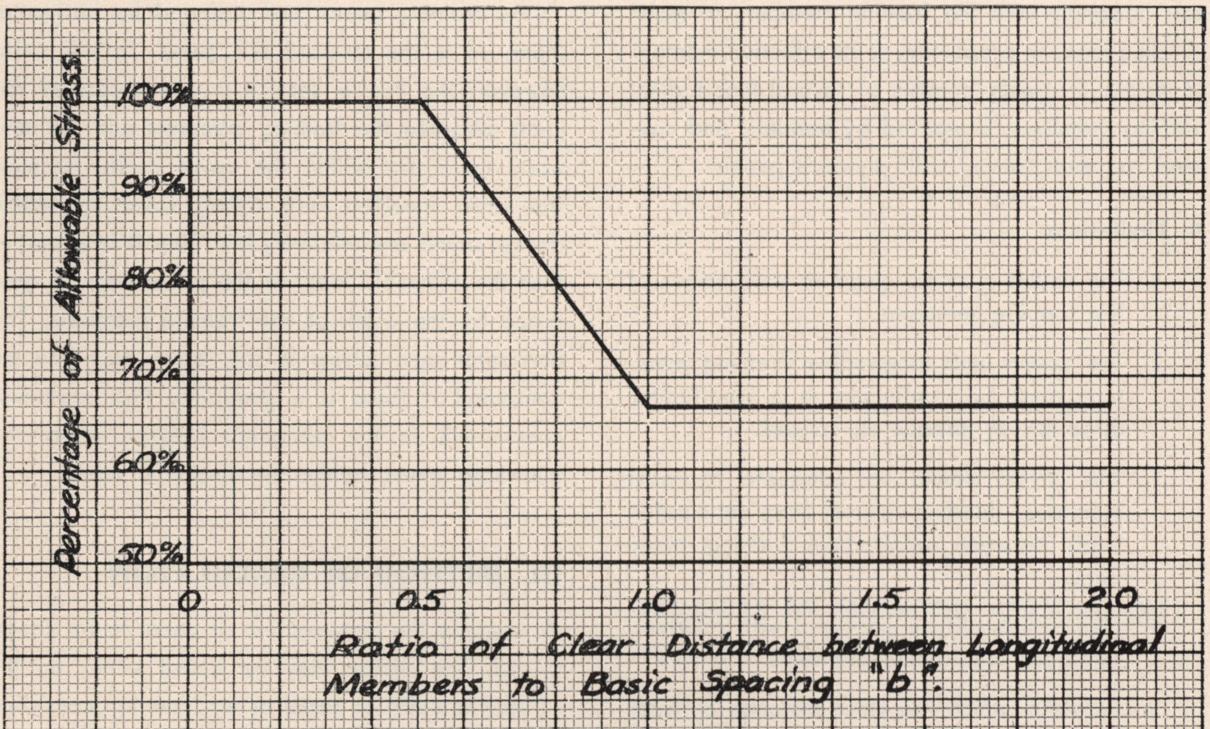


FIG. 1: Variation of Allowable Stress with Spacing of Framing Members.

TABLE I

RECOMMENDED WORKING STRESSES FOR DOUGLAS FIR PLYWOOD, psi

TYPE OF STRESS	GRADE OF PLYWOOD		
	EXT-DFPA•A-A	EXT-DFPA•A-B EXT-DFPA•Plyshield•AC	EXT-DFPA•Utility•B-C EXT-DFPA•Sheathing•C-C
DRY LOCATION (Moisture Content 16% or Less)			
Extreme Fiber in Bending	2190	2000	1875
Compression Parallel	1605*	1460*	1375
Rolling Shear (interior members)**	79	72	68
DAMP OR WET LOCATION (Moisture Content More than 16%)			
Extreme Fiber in Bending	1750	1600	1500
Compression Parallel	1285*	1170*	1100
Rolling Shear (interior members)**	79	72	68

*This stress is applicable to 3-ply plywood. For five or more plies, use stress for next lower grade.

**Refers to location of member with respect to adjacent framing members within panel.

G. The deflection under a given load may be calculated by using customary deflection formulas for the appropriate loading condition, employing the moment of inertia from paragraph D and the modulus of elasticity for the cover material (1,600,000 psi for Douglas fir plywood).

H. The allowable load as determined by rolling shear should be calculated for the interior framing member together with its contributing area of covers, because the critical shearing stress occurs in this region. The allowable shear may be found as follows:

1. Find the statical moment about the neutral axis of those parallel plies outside of the critical rolling shear plane. The critical shear plane lies within the plywood between the inner parallel ply and the adjacent perpendicular ply when the plywood cover has its face grain parallel to the longitudinal framing members. When the face grain of the cover is perpendicular to the framing member, the critical rolling shear plane lies between the inner perpendicular ply and the framing member. (See Section 2, page 2, Fig C.) The statical moment of the compression side is ordinarily larger, but if the tension side gives a higher value it should be used.

2. The allowable shear may be found from the formula

$$V = \frac{sIt}{Q}$$

V=Allowable external shear on section, lbs.

s=Allowable rolling shear stress, psi, taken from the table in paragraph F. (This value is independent of framing member spacing.)

I=Moment of inertia of the section, in.^{4}

t=Thickness of longitudinal member, in. (The stress will be concentrated above the framing member, even though the plane may occur within the plywood itself.)

Q=Statical moment of the parallel plies outside the critical shear plane, in.^{3}

The allowable shear load on the section should then be checked against the allowable bending load on the panel. The smaller value will determine the working load.

I. If desired, the allowable load as determined by horizontal shear at the neutral axis may be found from the formula

$$V' = \frac{s'It}{Q'}$$

V'=Allowable external shear on section, lbs.

s'=Allowable horizontal shear stress, psi, for the species and grade of the framing member.

Q'=Statical moment about the neutral axis of all parallel grain material above (or below) the neutral axis.

I and t have the same meaning as in paragraph 2.

TABLE 2.

BASIC SPACING "b" FOR VARIOUS PLYWOOD THICKNESSES

(For use in calculating the strength of stressed skin panels)

For three-ply plywood $b = 31h \sqrt{\dfrac{h}{\text{Total Thickness of Parallel Plies}}}$

For five or more plies $b = 36h \sqrt{\dfrac{h}{\text{Total Thickness of Parallel Plies}}}$

where h is the thickness of the plywood. A schedule of plywood constructions will be found on page 10, Section 2, "Designing with Plywood."

Plywood Thickness	**Basic Spacing "b," inches**	
	Face grain parallel to longitudinal members	Face grain perpendicular to longitudinal members
¼″ Sanded	10.35″	11.61″
$\frac{5}{16}$″ Rough	11.87	16.80
⅜″ Rough (3-ply)	14.25	20.13
⅜″ Sanded (3-ply)	16.43	16.43
⅜″ Sanded (5-ply)	18.10	20.25
½″ Rough and Sanded	23.25	28.5
⅝″ Rough and Sanded	29.1	35.6
¾″ Sanded (5-ply and 7-ply)	38.2	38.2
⅞″ Rough and Sanded	41.6	48.1
1″ Rough	45.5	58.9
1″ Sanded	54.5	47.9

SAMPLE CALCULATION OF A STRESSED SKIN PANEL

Suppose it is required to check the bending strength of the 4′ x 8′ floor panel shown in Fig. 2, when supported on a 7' 9" span. Assume Exterior Type Douglas fir plywood.

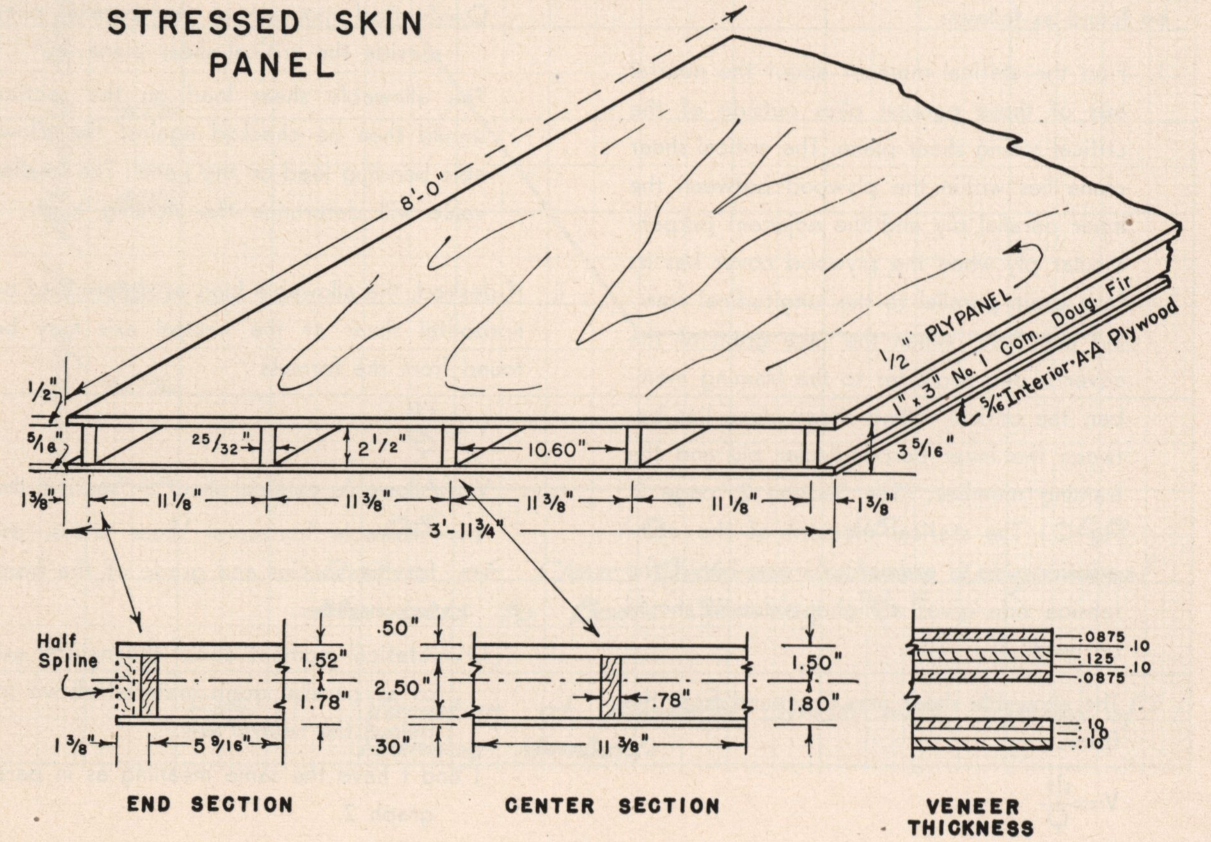


Fig. 2

Center Section:

1. **BASIC SPACING: (From Table 2)**

Top: $b = 23.25 > 10.60''$

Bottom: $b = 11.87 > 10.60''$

$\therefore$ Consider the covers to be the clear distance between joists plus the joist thickness, or $11\frac{3}{8}''$.

2. **NEUTRAL AXIS:**

Moments about base (neglect perpendicular plies)

		Area, in.^{2}	Moment, in.^{3}
Bottom:	11.38 (.2) (.15)	2.27	.34
Joist	.78 (2.5) (1.55)	1.95	3.02
Top	11.38 (.30) (3.05)	3.41	10.41
		7.63	13.77

N.A. lies $\frac{13.77}{7.63} = 1.80''$ above base.

3. **MOMENT OF INERTIA:**

The moment of inertia of the covers is most readily found from the parallel axis formula, $I = I_o + Ad^2$; that of the joist by using the formula for the moment of inertia of a rectangle about an axis through its base, $I = \frac{bh^3}{3}$. Moments of inertia of the covers about their own axes are listed on page 10, Section 2.

$$\text{Bottom: } \frac{11.38}{12}(.0294) + 2.27\,(1.65^2) = 6.22$$

$$\text{Joist: } \frac{.78}{3}(1.5^3 + 1.0^3) = 1.14$$

$$\text{Top: } \frac{11.38}{12}(.0926) + 3.41\,(1.25^2) = 5.43$$

$\therefore$ I for one beam section $= 12.79 \text{ in.}^4$

4. **ALLOWABLE STRESSES: (dry location)**

TOP

From Table I (and footnote under *) the recommended working stress for 5-ply EXT-DFPA•PLYSHIELD grade in compression parallel is 1375 psi.

Ratio of actual spacing to basic spacing is

$$\frac{10.60}{23.25} = .455 < 0.5$$

$\therefore$ Use full allowable stress of 1375 psi.

BOTTOM

Recommended working stress for A-A (So-2-S) grade in bending is 2188 psi. Ratio of actual spacing is

$$\frac{10.60}{11.87} = 0.89$$

$\therefore$ Reduce allowable stress to $74\% = .74\,(2188) = 1620$ psi.

5. **ALLOWABLE LOAD: (based on flexure)**

Resisting moments

$$\text{Top: } M = \frac{sI}{C} = \frac{1375\,(12.79)}{1.50} = 11{,}650 \text{ in.-lbs.}$$

$$\text{Bottom: } M = \frac{1620\,(12.79)}{1.80} = 11{,}500 \text{ in.-lbs.}$$

$\therefore$ Tension governs, and allowable uniform load on 7'-6'' span is

$$w = \frac{8\,(11{,}500)}{90\,(7.5)} \times \frac{12}{11.38} = 144 \text{ lbs./ft.}^2$$

6. **ROLLING SHEAR:**

Statical Moment of Top

(This is computed for the two upper plies only, because when the face grain of the plywood is parallel with the joist, as in this case, the critical rolling shear plane falls within the plywood, between the lower parallel and perpendicular plies.)

$$Q = (.2125)\,(11.38)\,(1.34) = 3.24 \text{ in}^3.$$

Allowable shear on panel (determined by rolling shear stress of 72 psi for EXT-DFPA•PLYSHIELD grade).

$$V = \frac{sIt}{Q} = \frac{72\,(12.7)\,(.78)}{3.24} = 221 \text{ lbs.}$$

Allowable load to cause shear of 221 lbs.

$$= \frac{2\,(221)}{7.5} \times \frac{12}{11.38} = 62 \text{ lbs./ft.}^2$$

7. **HORIZONTAL SHEAR:**

The maximum horizontal shear stress in the joists at the neutral axis may be checked by a similar process.

Statical moment of all parallel grain material above the neutral axis

$$Q' = 3.41\,(1.25) + 1.0\,(.5)\,(.78) = 4.65 \text{ in.}^3$$

Allowable shear on panel (governed by an assumed shear working stress for lumber of 120 psi)

$$V' = \frac{120\,(12.79)\,(.78)}{4.65} = 256 \text{ lbs.}$$

Allowable load to cause shear of

$$256 \text{ lbs.} = \frac{2\,(256)}{7.5} \times \frac{12}{11.38} = 72 \text{ lbs./ft.}^2$$

8. **DEFLECTION:**

(a.) The uniform load required to cause a deflection of 1/360th of the span is found from the usual deflection formula.

$$d = \frac{90''}{360} = \frac{5}{384}\,\frac{Wl^3}{EI} = 0.25''$$

The value of E is taken as that of the covers, and is 1,600,000 psi for Douglas fir plywood, regardless of grade and location.

$$W = \frac{384\,(1{,}600{,}000)\,(12.7)\,(.25)}{5(90^3)} = 535 \text{ lbs.}$$

where W = load on I-section 11.38" wide and 7'6" long.

$$\therefore w = \frac{535}{7.5} \times \frac{12}{11.38} = 75 \text{ lbs./ft.}^2$$

(b.) The load required to cause a deflection of the top cover only of 1/360th of the span between joists may be found from the deflection chart, page 10, Section 4. Taking the joists at 12" centers, and noting that the grain of the face plies is perpendicular to the span, it can be seen that the continuous beam load to cause a deflection of 1/360th of the span is

$2(215)\,(.40) = 172 \text{ lbs./ft.}^2$

Evidently the maximum load on the panel will be limited by its shear strength, rather than its bending strength. A check on the end section reveals that there, also, rolling shear is the limiting factor, when the reduced allowable shear stress caused by unsymmetrical stress concentration is taken into account (see Table 1, p. 6, Section 2).

It is interesting to compare these design values with actual tests on three panels constructed commercially by a house prefabricator for use as flooring. Tests were conducted by the National Bureau of Standards, Washington, D. C., and are reported in their Building Materials and Structures Series, Report BMS 104, pp. 22-24, as Floor Panel DX.

The test cross section was identical with that in the calculation, except that it included half-splines on each side, to conform with the actual construction. It is doubtful that these 1" x 3" half-splines contributed appreciably to the rolling shear strength, as they were not glued to the covers.

Three panels were tested on a 7½ foot span under quarter-point loading; they sustained maximum loads of 323, 320, and 320 lbs. per sq. ft., respectively, with first failures apparently caused by rolling shear. The loads at the allowable deflection of 1/360th of the span averaged 95 lbs. per sq. ft., rather than 75 lbs. as computed.

When the design load of 60 lbs. per sq. ft. is compared with the ultimate of 321 lbs. per sq. ft., and the factor of safety of 5⅓ contrasted with the usual factor of 3.6 on the ultimate for wood, it might be concluded that the allowable rolling shear stresses are conservative in some cases. It seems reasonable to suggest that safe design loads could be based upon tests when possible, provided that the quality of the assembly is guaranteed by inspection or certification.

In the case at hand, for example, the safe load on the panel could be at least double the usual floor design load of 40 lbs, per sq. ft.

DOUGLAS FIR PLYWOOD ASSOCIATION

Tacoma 2, Washington

SECTION
NINE

Revised November 1, 1948

THE DESIGN OF BUILT-UP BEAMS WITH PLYWOOD WEBS

INTRODUCTION

This section outlines a rational method of designing plywood beams to withstand given loads with adequate safety factors. For convenience in the following discussion the term "plywood beams" will be used to designate built-up wood beams having plywood webs.

DESCRIPTION

Plywood beams generally consist of one or more vertical plywood webs to which lumber flanges are glued along the top and bottom edges. At intervals along the beam vertical lumber stiffeners separate the top and bottom flanges.

Although almost any species of wood may be employed for the plywood webs, the practice to date has been to use Douglas fir plywood for most built-up construction, because of its relatively high strength, low cost, ease of working and ready availability. Therefore, in the following discussion all plywood design values given are those for Douglas fir. Similarly any species of wood may be employed for the lumber flanges but practical considerations usually limit the choice to those commonly available in structural grades.

Although there is no definite limitation on the span length which may be bridged with plywood beams, it does appear that they will be economical for spans up to 60 feet. The beams may range in depth up to 48″ utilizing stock widths of plywood, or even deeper if extra width panels are ordered.

The shape may be an I-beam with a single web and flanges top and bottom, or it may include any number of webs separated by flanges, the outer layers being either webs or flanges, in which case the assembly is generally called a box beam. In either case, the design procedure is similar.

APPLICATIONS

Plywood beams have been used in frame buildings as floor and roof girders. A similar form may be employed advantageously as a spaced column. They have been used for bridges, to support crane runways, and as lintels in building walls.

Plywood girders appear to offer the following advantages:

1. Large units may be fabricated.

2. Flanges may consist of comparatively thin members which can be more readily seasoned and are more readily available than comparable solid members.

3. Maintenance such as bolt-tightening is minimized.

4. High-strength material may be placed at points of greatest stress and lower grade material elsewhere.

5. Economy of material can be obtained because:

   a. The cross-section along the beam may be varied to conform to the particular load requirements.

   b. Material is placed most advantageously to resist stress.

   c. The high allowable shear strength of plywood permits the use of relatively thin webs.

6. Deep stiff sections may be built economically.

7. The members may be built so as to form a variety of shapes, varying from straight or slightly cambered, to an arch, if the flange pieces are suitably laminated.

8. The light weight of the members facilitates transportation and erection.

9. The member performs as an integral unit, requiring no consideration of secondary stresses, except, perhaps, where discontinuous webs are used. (See page 14.)

10. As only dry wood is used, the strength and stiffness are increased, and the shrinkage minimized.

CHOICE OF SECTION

The design of a plywood beam yields readily to ordinary engineering analysis although the usual formulas are modified slightly in certain respects. Knowing the span and loads, it is possible to select a trial cross-section as shown subsequently. This trial section may then be checked for flexure, horizontal shear, shear between flange and web and deflection. Design of splices and location of vertical stiffeners and lateral supports may also be checked. If the member has a variable cross-section, each section should be investigated. Obviously the form and dimensions of the members should be such that it can be built with available facilities.

Before selecting the trial cross-section, the type of beam—whether I or box—constant or variable section, should be determined. In general an I-beam will be suitable only for the shorter spans and lighter loads—i.e., lower shear requirements, unless thick plywood is obtained. An I-beam is somewhat simpler to build and requires fewer vertical stiffeners. On the other hand the box beam employing two or more webs has greater lateral stability, greater design flexibility in that it will allow more variation in the cross-section, provides space within the beam for concealing utilities such as electrical conduits and can be designed to provide a smooth, clean external surface.

The advantages of employing a variable cross-section become more apparent as the span increases, particularly beyond the commonly available lengths of lumber. If full-length lumber is not obtainable for the flanges, then they must be built up from a number of pieces, thus providing an opportunity to vary the amount of material to conform with the bending moments. Also the longer spans provide better opportunity to alter the number of webs so as to conform with the varying shear requirements. However, in this latter respect it should be noted that secondary stresses do occur where there is a change in the number of webs. Nevertheless, six full-size test beams having discontinuous webs showed no indication that such stresses reduced their ultimate loads. In addition, the depth of the beam may be varied to produce a curved upper surface—which may be desirable for roof drainage, for example—or camber in the lower flange.

Even a short span plywood beam lends itself to a varying cross-section when weight reduction is a prime consideration.

Gluing

All joints between lumber and plywood can be made satisfactorily with glue. Approximately the full strength of wood parallel to the grain can be developed with glued end-grain scarf joints, whereas glued side-grain butt joints are sufficient to develop the full strength perpendicular to the grain. Adequate gluing procedure requires proper equipment, as well as knowledge and experience. Fortunately there are numerous fabricators throughout the country today who possess such prerequisites.

A number of high grade assembly glues are available today, but they should be used in strict accordance with manufacturer's directions, and with special attention to temperatures, assembly times, and spreads. Fully as important are moisture content and surfacing of lumber, method of applying pressure, and time and conditions of cure.

A variation of shop-gluing technique which has been practiced to some extent is nail-gluing, a method which provides a member having the advantages of a glued-up beam, but which requires no special equipment to fabricate; however, the requirement of knowledge and experience on the part of the fabricator is still of paramount importance. With this method the beam is ordinarily built on or near the site, using nails to provide pressure for setting the glue. It is argued that the nails provide some insurance against total failure should the glue bond be faulty at any point. This method generally permits less accurate control than shop fabrication in many respects, and probably causes a small strength reduction compared to shop-glued beams due to the presence of nails in the tension flange and of butt joints in the section. A nail-glued bond may not be adequate for severe exposure conditions.

It is customary to design with individual laminae which are fairly thin—usually 1″ or 2″—because of the necessity of using uniformly dry lumber and the requirements of sufficient flexibility for intimate contact at the glue line between adjacent members.

DESIGN METHOD

Trial Cross Section

After deciding upon the type of beam and the method of manufacture, the next step is to select a preliminary cross-section. If the depth is not limited by such factors as headroom requirements or press dimensions, it may be selected by combining the flexure and deflection equations, and solving for the span-depth ratio. It is necessary only to know the span, loads, allowable fiber stress and deflection and the elastic modulus. This method assumes that the moment of inertia is constant, but this factor may be compensated for by suitable approximations. It is ordinarily economical to select a depth equal to a stock width of plywood in order to reduce cutting and waste.

Next, the required web thickness at a given section may be determined approximately from the formula

$$t = \frac{1.25V}{hv} \qquad (1)$$

where

t=total web thickness, inches.

V=shear at the section, lbs.

h=beam depth at the section, inches.

v=allowable plywood shear stress perpendicular to the plane of the plies, psi.

The value of "t" thus found is useful in a preliminary design, but it should be tempered by a consideration of the available plywood thicknesses and of the number of contact surfaces necessary to develop the flange-web shear.

The approximate minimum flange depth required at the section of maximum shear will generally be about six times the thickness of the adjoining web if the web has a flange glued to it on only one side, or three times the web thickness if there are two flange faces as in an I-beam. These approximate flange depths are those required by the allowable shear on a glued joint; they do not necessarily indicate a desirable depth at the section of maximum bending moment.

The next step is to ascertain the moment of inertia requirements for both flexure and deflection. Use may be made of the well-known flexure formula (modified by a form factor as discussed in Section Seven), giving the equation

$$I = \frac{Mc}{Ff} \qquad (2)$$

where

I=the net moment of inertia, inches^4, of the total section neglecting any flange laminations which are butt-jointed at the given section, and those plies in the web having their grain perpendicular to the span.

M=the bending moment at the section, inch-lbs.

c=distance from the neutral axis to the extreme fiber, inches.

F=form factor, assumed as 0.80 for preliminary purposes.

f=allowable fiber stress in bending of the flange lumber, psi.

The allowable deflection may dictate a minimum value for the average gross moment of inertia. (The gross moment of inertia includes all fibers, regardless of butt joints, with grain parallel to the span.) The customary deflection formulas may be employed, but a factor should be added to provide for the effect of shearing deflection, which is not negligible in built-up plywood beams. Again for preliminary purposes only, it will be sufficiently accurate to add 25% to the deflection as calculated from the customary formulas. If the cross-section is variable along the span the moment of inertia thus found will be the average effective I for the entire beam.

When I has been determined, the flange requirements may be estimated by subtracting the I of the webs at the section in question. The flanges may be made up from standard lumber sizes, but allowance should be made for resurfacing which will reduce the thickness of the individual pieces after scarfing, and the width or depth of the flanges before they are glued to the web.

Accurate Design Check

It should be emphasized again that up to this point the sections obtained are purely for trial purposes. Formulas suggested are only approximations, to fit average conditions. The design should not be considered satisfactory until the sections have been carefully inspected by the methods outlined in the following paragraphs.

1. FLEXURE

When investigating the bending strength of a section the flexure formula may be used as in a previous paragraph (Equation 2), but values of the moment of inertia and of the form factor should be computed from the actual net dimensions.

In computing the net I it should be reiterated that only those plies in the web having their grain parallel to the span are considered. Theoretically the perpendicular plies, about one-twentieth as effective in bending as the parallel, also contribute, provided they are edge-glued. The error in omitting them from the calculations however, usually amounts to less than one-half of one percent and is on the conservative side. The constructions of various plywood thicknesses may be found in Section Two, page 9. Any openings in the flange, such as butt joints, near the section in question should be omitted in calculations of the net moment of inertia. However the location of the neutral axis is determined by the gross section, exclusive of perpendicular grain material, and is not affected by local irregularities such as butt-joints. Thus the bending stresses will be calculated with the net moment of inertia and the distance of the fiber from the neutral axis of the gross section.

Values of the form factor, which depend on the relative dimensions of the section, may be taken from the chart on page 3, Section Seven. This chart, based on data published by the United States Forest Products Laboratory, is a graphical solution of their equation for the proportional limit form factor. Various species will have slightly different constants in this equation, but the error caused by using the average constant is negligible.

In Section Seven is discussed the reason why a form factor is applied to I and box beams and a depth factor to rectangular beams. It is also pointed out that for the compression flange, working stresses in bending should be increased by 10/9ths when a form factor is employed, or more conveniently, the form factor itself may be multiplied by 10/9ths.

The actual stress existing within either tension or compression flange is given by the equation

$$s = \frac{Mc}{I} \tag{3}$$

whereas the form factor applies only to the compression flange. Thus the compression flange will govern the design in a beam that is symmetrical about its neutral axis, while the tension flange will be understressed.

This suggests a possibility for utilizing material more closely, either by reducing the dimensions of the tension flange, as is commonly done in aircraft wing beams, or by using a lower grade of lumber there. This latter may be accomplished by specifying, for example, a Select Structural grade of Douglas fir for the tension flange, and a Dense Select Structural grade for the compression flange. Each flange is then checked separately for bending.

2. HORIZONTAL SHEAR

The horizontal shear stress in the plywood web may be checked at the sections of maximum shear by applying the formula

$$v = \frac{VQ}{It} \tag{4}$$

where

v = horizontal (or vertical) shear stress on the plane under consideration (usually the neutral axis), psi.

V = total shear on the cross-section, lbs.

Q = statical moment about the neutral axis of all fibers whose grain is parallel to the beam's axis and lying outside of the plane under consideration, inches^3.

I = gross moment of inertia of all parallel fibers about the neutral axis, inches^4.

t = total thickness of the plywood webs at the cross-section, inches.

It is inaccurate, and not on the side of safety, to assume that the shearing stress is evenly distributed throughout the depth of the plywood webs, as is done in the case of steel I-beams, because the relative proportions of web and flanges are different for beams of the two materials. It is also inaccurate and unduly conservative to use the customary formula for rectangular wooden beams, $v = \frac{3V}{2A}$, as the latter is simply a special case of Equation 4, and does not apply to webbed beams.

3. FLANGE-WEB SHEAR

Where the flanges and webs are joined by glue, the joint is parallel with the plane of the plies and its strength is limited by rolling shear. The allowable stress is considerably lower for rolling shear—that is, shear in the plane of the plies—than for shear perpendicular to the plane of the plies.

The outline below suggests a method for determining the maximum shear stress existing between the flange and web. It is based on the assumption that the horizontal shear stress is equal in all webs, and that they therefore receive load from the flanges in proportion to their thicknesses. This assumption in turn requires that the flange areas be so proportioned that they are not utterly inconsistent with the thicknesses of the adjoining webs. No definite rules can be laid down, as such proportioning is simply a question of proper design judgment.

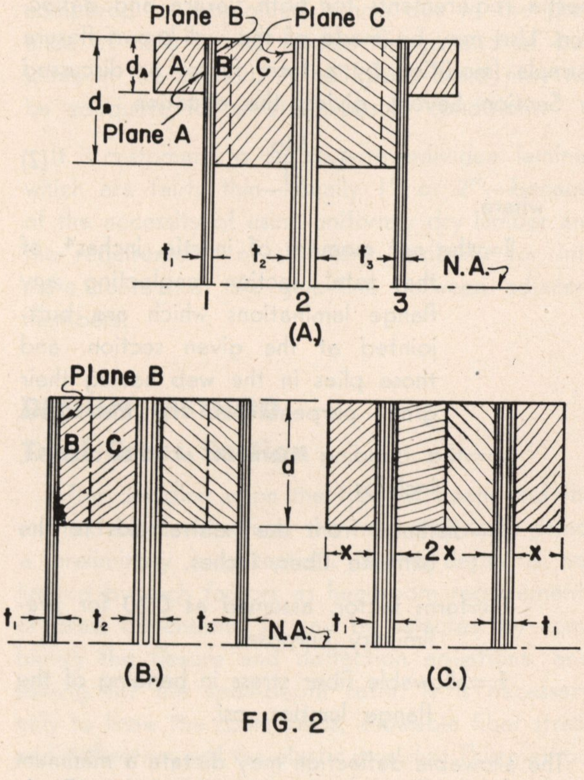


FIG. 2

In Figure 2A let Q_A be the statical moment about the neutral axis in inches^3 of flange area A, Q_B of flange area B, etc. Also $Q_1 = Q_A + Q_B$ = the statical moment of that part of the flange area which contributes to the horizontal shear in web 1. Q_2 and Q_3 have similar relations for webs 2 and 3. Then $Q_1+Q_2+Q_3=Q\ fl$. Also

$$Q_1=\frac{t_1\,(Q\ fl)}{(t_1+t_2+t_3)} \tag{5}$$

from the assumption that the horizontal shear stresses in all webs are equal, neglecting the statical moments of the plywood webs themselves.

Then the flange-web shear stress on plane A is

$$s_A=\frac{V\,Q_A}{Id_A} \tag{6}$$

where V and I have the same meanings given in Equation 4, and d_A is the flange depth in inches indicated in Figure 2A. Similarly

$$s_B=\frac{VQ_B}{Id_B} \tag{7}$$

where $Q_B=Q_1-Q_A$.

It is apparent that this process can be extended to as many planes as is necessary to determine the maximum shear stress.

A special case, common in practice, occurs when plywood webs form the outside layer of the beam (Fig. 2B). Here Q_A is zero, and $Q_B=Q_1$. Then

$$s_B=\frac{VQ\ fl\ (t_1)}{Id\ (\Sigma t)} \tag{8}$$

where Σt=the total web thickness, inches.

Another special case, which also occurs frequently, exists when the webs are of equal thickness, and the flanges of equal depth and so divided as to give an equal stress intensity to each contact face. Such a condition will hold in an I-beam or a box beam such as that shown in Fig. 2C. In this case the flange-web shear stress is

$$s=\frac{VQ\ fl}{nId} \tag{9}$$

where n is the total number of contact faces between the webs and flanges.

If the face grain of the plywood web is parallel to the span, as it usually is, the value of Q in all of the above formulas should theoretically include the thickness of the face veneers of the plywood for a depth equal to that of the flange, as in this case the critical rolling shear plane lies within the plywood itself between the face and its adjacent veneer. Face veneers can be safely neglected in practice, however, because their horizontal shear strength in such a case will be added to the rolling shear strength of the joint.

Localized flange-web shear stresses of undeterminable magnitudes are produced when loads are transmitted into the beam from adjacent framing members which bear directly on the lower flange. The stress distribution is further complicated by the overturning effect of the eccentrically applied load. For these reasons it is not recommended that loads bear directly on the lower flange.

4. DEFLECTION

Plywood girders, if the webs are effectively spliced, will act as a unit and deflect according to elastic theory. Consequently the usual beam deflection formulas may be employed. However, these formulas neglect the effect of shearing deformation which is ordinarily ignored for rolled steel sections, although for deep steel plate girders it would not be unusual for the shear deflection to equal 15% of the bending deflection. In plywood beams, shear may account for a considerably higher percentage of the total deflection. One reason for this fact is that in wood the shearing modulus is about 1/16th of the true elastic modulus, while in steel it is about 2/5ths of the elastic modulus. Consequently the shear deflection of a plywood beam should be calculated and added to the bending deflection.

The amount of shear deflection is affected by the position and intensity of the loads, the span, cross-sectional dimensions and the shear modulus of the webs. Formulas for computing shear deflection have been worked out theoretically and verified experimentally by the United States Forest Products Laboratory (Reference 1).

The shear deflection of a section which is symmetrical about its horizontal axis may be expressed by the formula

$$d_s=\frac{PlKh^2C}{GI} \tag{10}$$

where

d_s=shear deflection, inches.

P=total load on beam, lbs.

l=span length, inches.

K=a factor determined by the beam cross-section, and shown in Fig. 3.

h=depth of beam, inches.

C=a coefficient depending on the manner of loading, also shown in Fig. 3.

G=shearing modulus of the webs, psi.

I=gross moment of inertia of the section, inches^4.

For sections symmetrical about their neutral axis, the factor K is determined by the same ratios that affect the form factor, i.e., d/h, the ratio of the compression flange depth to the beam depth, and t_1/t_2, the ratio of the total web thickness to the total beam width.

The shearing modulus of Douglas fir plywood having the face grain either parallel or perpen-

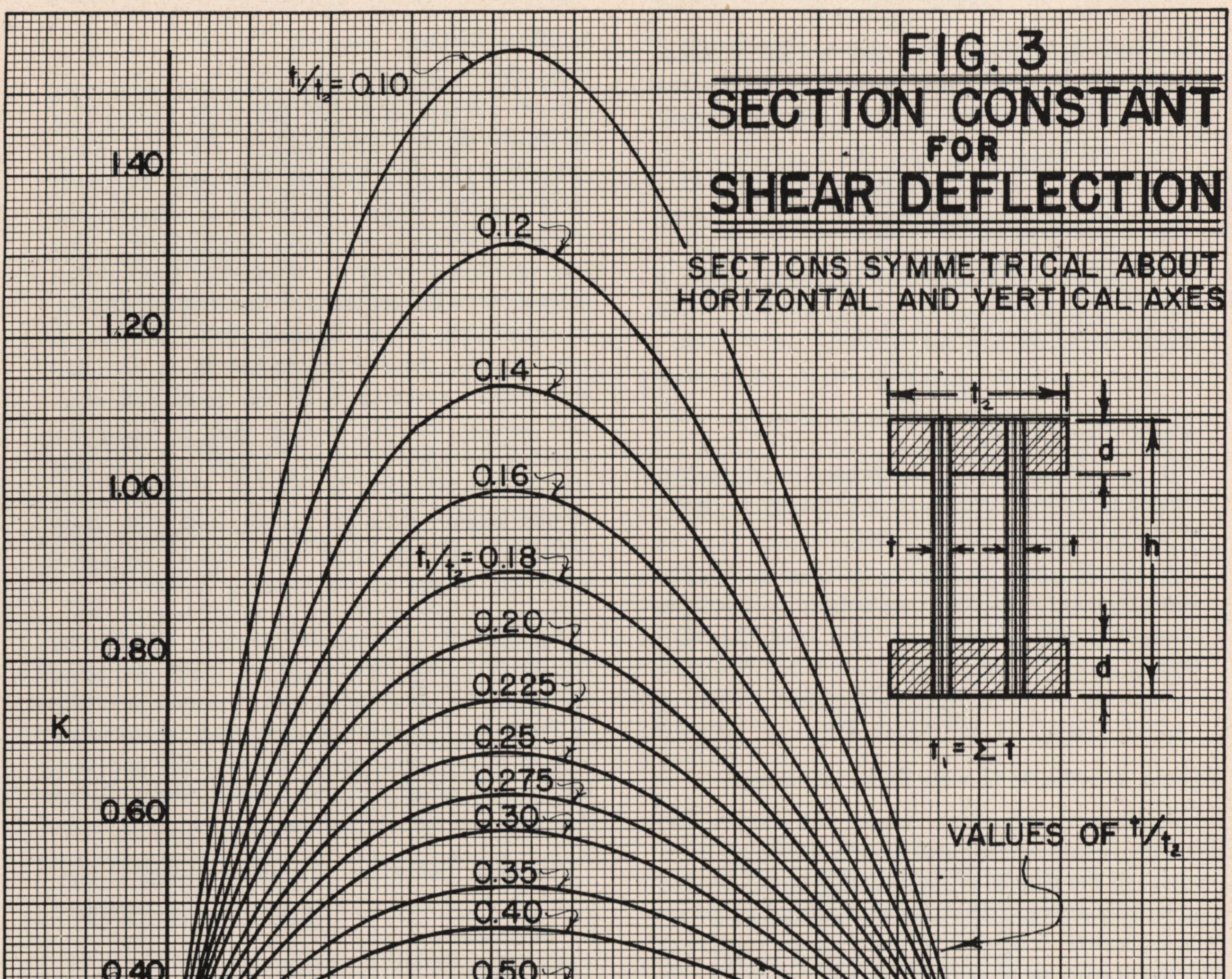


FIG. 3
SECTION CONSTANT FOR SHEAR DEFLECTION

SECTIONS SYMMETRICAL ABOUT HORIZONTAL AND VERTICAL AXES

$$K = \frac{1}{4}\left[1 + \frac{12d^3 - 18hd^2 + 6dh^2}{h^3}\left(\frac{1}{(t_1/t_2)} - 1\right)\right]$$

$$D_s = \frac{PlKh^2C}{GI}$$

D_s = Shear Deflection
G = Shear Modulus
I = Moment of Inertia

dicular to the span may be taken as 117,000 psi at 15% moisture content. If the face grain is at 45° to the span, a value of 375,000 psi corresponds to a 15% moisture content. These values are derived from information issued by the Forest Products Laboratory.

If the cross-section of the beam varies along its length, the dimensions used for calculating the factor K, as well as the depth and moment of inertia in the shear deflection formula should be those of the sections subjected to shear, weighted with respect to the area of the shear diagram. For instance if third-point loading is used, the calculations are made for the average cross-section in the outer thirds.

If a beam is subjected to two loadings simultaneously—e.g., a uniform and a concentrated load—the shear deflection for each loading condition may be computed separately and added.

In computing the deflection caused by bending the true modulus of elasticity of the flange species should be used in the ordinary deflection formulas. The "true modulus" is simply a value 10% higher than the ordinary modulus of elasticity for the given species as listed in various handbooks and the grading rules of lumber associations, inasmuch as the ordinary moduli for wood contains an allowance of 10% to provide for shearing deflection occurring in a solid rectangular beam having a span-depth ratio of about 14. This approximate shear correction is not required when the accurate correction described above is made.

It should be noted that if a beam is glued, the lumber should be initially dry, and the elastic moduli will accordingly be higher than for green lumber of the same species.

5. LATERAL STABILITY

Plywood beams are necessarily deeper than solid lumber beams of equal strength and width. Because the member is ordinarily relatively deep by comparison with its overall width, it is desirable to give some attention to its lateral stability. A plywood beam will have a lower critical buckling load than a solid wood beam of the same species, moisture content, and overall dimensions. Furthermore, a box beam possesses greater lateral stability than an equally strong I-beam having the same overall depth and flange depth, because the former type possesses a higher torsional constant.

This does not mean that plywood beams are unstable laterally. In fact, one box beam tested under third-point loading on a 24-foot span showed no lateral buckling at an ultimate of almost five times its design load, although its overall cross-sectional dimensions were $6\frac{7}{8}''$ x 30″ and it was unsupported laterally. The ratio of the moment of inertia about its horizontal axis to that about its vertical axis at the center section was 27.1.

Nevertheless it is desirable to provide lateral support systematically, and to have some method of gaging its effectiveness. To this end it is suggested that the upper flange be considered as a column that tends to deflect sidewise between points of support. Then the bending stress in the compression flange will depend upon the ratio of the distance between supports to the flange width, in the same way that the allowable stress in a wood column is varied with its l/d ratio, to minimize the possibility of buckling.

The wood column formulas themselves may be used to obtain the percentage of allowable bending stress permissible in the compression flange. These values vary with the species and grade of flange lumber. For example, Fig. 5 shows the percentages of allowable bending stresses for various ratios of unsupported length to flange width for lumber having a modulus of elasticity of 1,600,000 psi and allowable compression stresses parallel to the grain ranging from 900 to 1600 psi. Similar curves can be readily constructed for other values of the elastic modulus.

The curves may be used in determining points for attachment of lateral bracing when the stress is known, or vice versa.

As an example, suppose it is known that the distance between lateral supports of a given beam will be 12 feet in the region of maximum moment, and that the width of the flange is 10″. The flange will consist of Select Structural Douglas fir lumber, with a modulus of elasticity of 1,600,000 psi, allowable bending stress 1600 psi, and allowable compression stress parallel to the grain 1200 psi. Then $l/w=144/10=14.4$. Reference to the 1200 psi curve shows the percentage of allowable bending stress to be 95%. Then the stress to be used in the flexure formula is $.95(1600)=1520$ psi. It should be noted that this reduction will be in addition to that caused by the form factor.

The Forest Products Laboratory has developed formulas expressing the critical buckling loads of various sections, including solid rectangular beams and I-beams, under a number of conditions of restraint and loading (Ref. 4 and 5). These formulas serve as a valuable guide in some cases. Their application, however, may become rather complicated when the section varies along the span, or becomes more complex than a simple box beam.

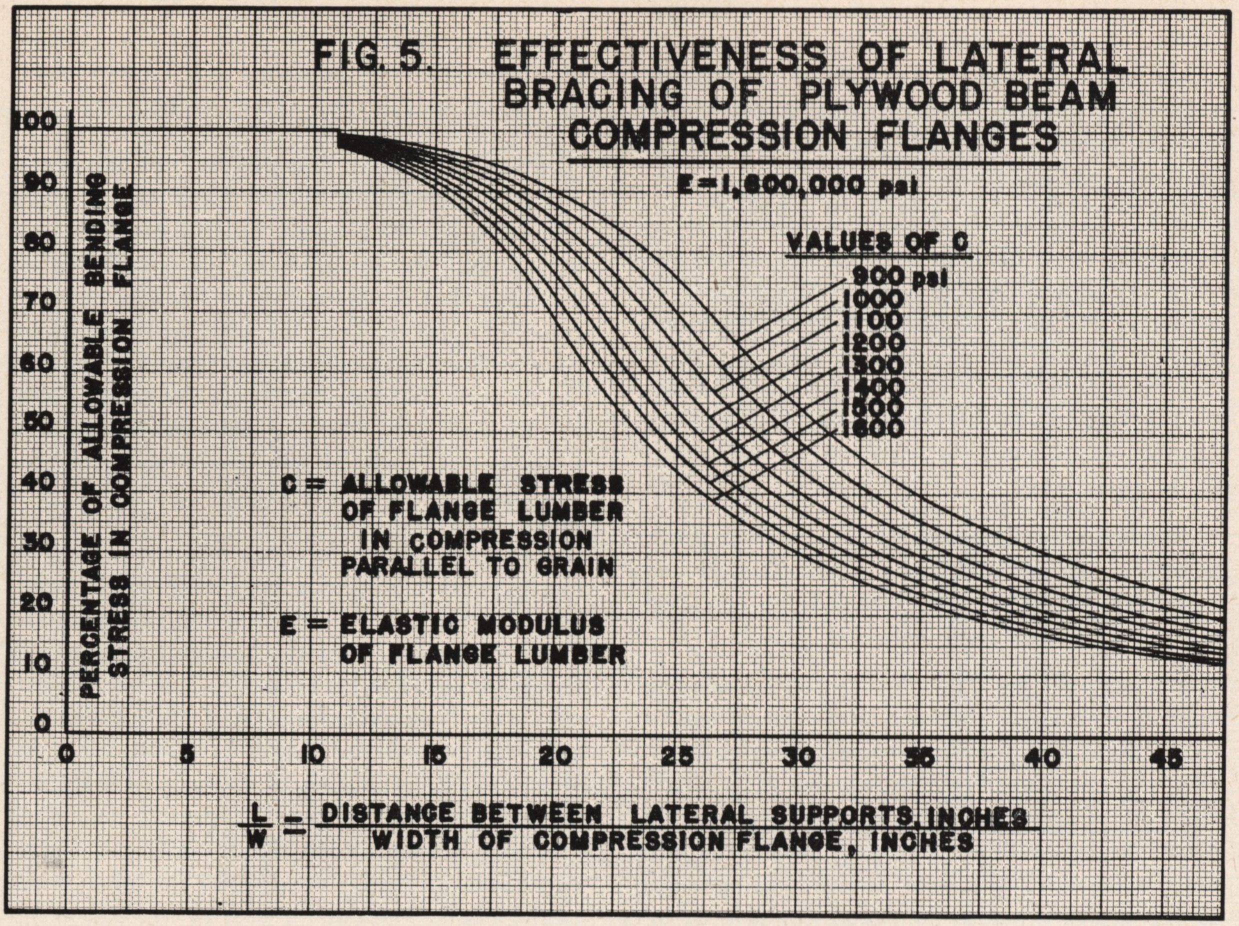


FIG. 5. EFFECTIVENESS OF LATERAL BRACING OF PLYWOOD BEAM COMPRESSION FLANGES

6. STIFFENERS

Plywood beams usually require several vertical stiffeners glued to the webs between inner surfaces of the top and bottom flanges. The function of these stiffeners is to stabilize the web against shear and buckling, to separate the flanges under loading, to distribute loads into the beam at the point of application and to space the flanges accurately during the fabrication process.

From the standpoint of design they may be divided into two classes—those which distribute load at points of heavy concentration, such as reactions, and those which are required principally to stabilize the web. One stiffener, of course, may serve both purposes.

The location of load-distribution stiffeners will be determined by the placement of the reactions and of heavy concentrated loads. Such stiffeners may be designed on the allowable bearing strength of the flange lumber in compression perpendicular to the grain. If the width of the stiffeners is made equal to the full flange width, any eccentricity in the load application to the flange will not place added stress on the flange-web joint due to an overturning effect. In such a case the minimum stiffener dimension parallel to the span will be

$$x = \frac{P}{w\,c\perp} \qquad (11)$$

where

x = stiffener thickness, inches.

P = concentrated load or reaction, lbs.

w = total beam width, inches.

$c\perp$ = allowable bearing strength of the flange lumber perpendicular to the grain, psi.

This formula will be conservative, providing the load is evenly distributed across the full beam width, because there will be some longitudinal stress distribution along the beam.

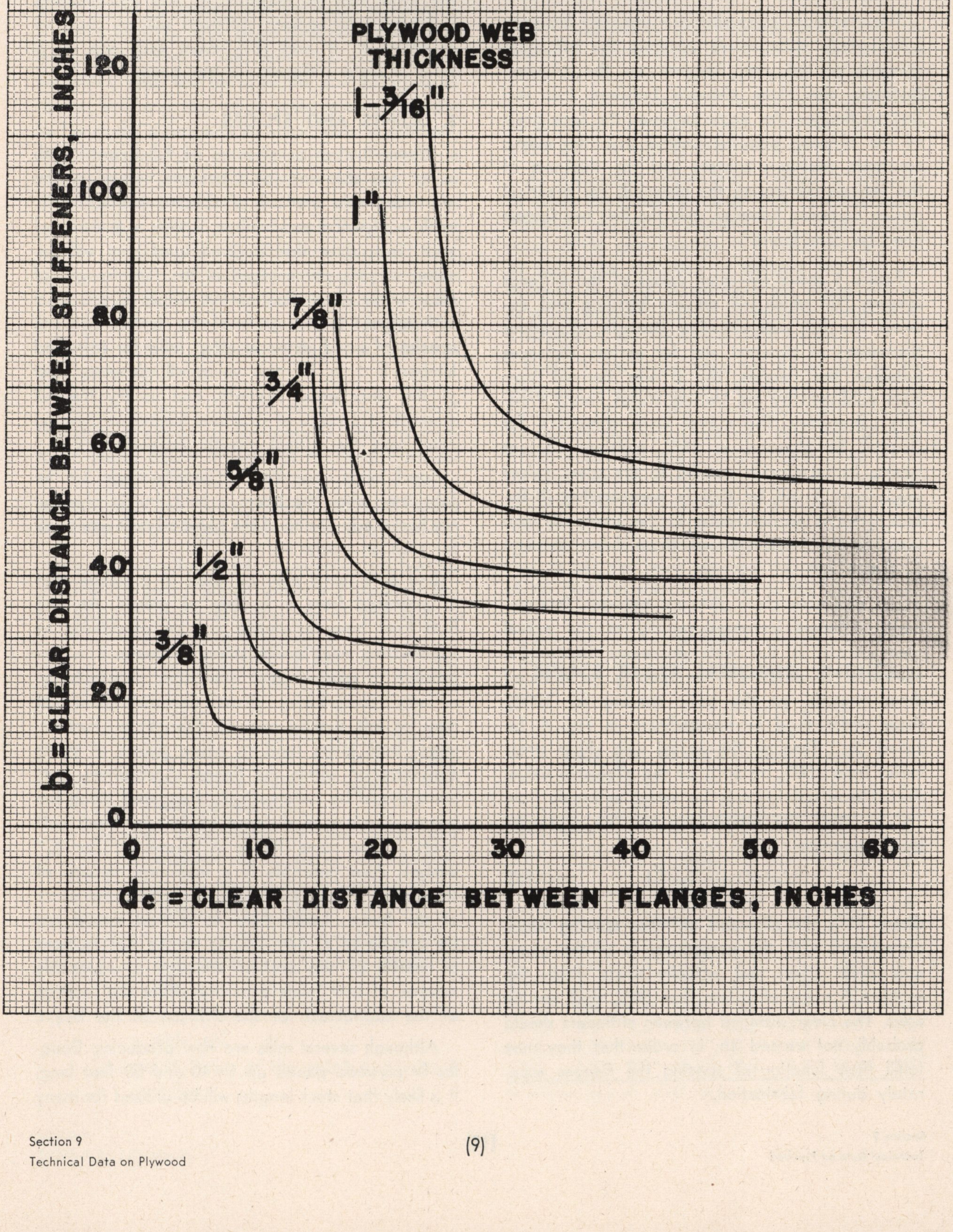
Complete design information on the size of intermediate, or web-stabilizing stiffeners is not available. In general, their cross-sectional areas should provide adequate column strength to resist any compressive forces from the flanges and to provide sufficient bearing area. They should also be thick enough to give sufficient glue area against the web to prevent the latter from buckling. Although the selection of an adequate intermediate stiffener must be left to the judgment of the designer, the following remarks may be of assistance:

1. Their width should be equal to the flange width.

2. Ordinarily they need not be as thick as load-bearing stiffeners for the same beam.

FIG. 4

BASIC INTERMEDIATE STIFFENER SPACING FOR PLYWOOD BEAMS



3. It is convenient that they be of solid rectangular cross-section and of some readily available thickness. A suggested minimum is $\frac{3}{4}''$.

4. To reduce the possibility of buckling it is suggested that the thickness be at least one-sixth of the outstanding dimension if only one edge of the stiffener is glued to the web (Ref. 5, page 9). With this requirement, for example, a $\frac{3}{4}''$ stiffener would be adequate for a beam having a flange width of $4\frac{1}{2}''$ or less.

7. WEB BUCKLING

The maximum clear distance required between intermediate stiffeners to maintain the full shear strength of the web depends on the plywood thickness, the clear distance between top and bottom flanges, the angle between the face grain of the plywood and the beam's axis, the elastic constants, moisture content and the thickness of the veneers composing the plywood. For practical design purposes, however, only the first two factors need be considered.

Figure 4 shows maximum values of the stiffener spacing "b" which will prevent buckling and develop the full web shearing strength of various thicknesses of plywood and for various distances between the flanges, d_c. It was derived from data published by the United States Forest Products Laboratory. The curves were computed for Douglas fir plywood at 15% moisture content having its face grain parallel to the span; values read from it will be slightly conservative if the face grain is perpendicular to the span. The plywood constructions listed are those shown on page 12, Table 2.

The allowable clear distance between intermediate stiffeners increases rapidly as the actual shear stress in the web decreases below the allowable. To determine the spacing for reduced shear stresses, let the basic spacing for 100% shear stress, read as the ordinate in Fig. 4, be called "b." Then as the actual web shear stress varies from 100% to 50% of the allowable value, the stiffener spacing may be varied uniformly from "b" to "3b." This relationship may be expressed by the formula

$$b' = b\left(1 + \frac{100 - p}{25}\right) \qquad (12)$$

where b' is the allowable stiffener spacing which corresponds with the percentage p of the allowable shear stress that actually exists within the section. A value of p less than 50% shall not be used. The clear distance between stiffeners should probably not exceed 3b, in order that they may fulfill their function of spacing the flanges accurately during fabrication.

The average plywood beam will not require many stiffeners because the span lengths usually are not excessive, so that the length of beam subjected to high shear stresses will not be great. Furthermore, the shear requirements are such that the webs used will probably be moderately thick. For example, a 40′ beam of constant section, subjected to a uniform load, will have at the most only 20′ of its length stressed to 50% or more of the allowable shear value. If the smallest web used is $\frac{3}{4}''$, "b" will not be less than 3′. Simple calculations based on the foregoing paragraphs will show that in addition to the end bearing stiffeners, intermediate stiffeners will be required at points 3′, 8′ and $15\frac{1}{2}'$ from each end, or a total of eight stiffeners for the beam.

8. JOINTS AND SPLICES

Where lumber or plywood of full beam length is not available end joints become necessary. The most desirable method of splicing longitudinally, both in plywood and lumber, is with a glued scarf joint. Tests show that a glued scarf joint will develop varying percentages of the full strength of clear wood depending on the slope of the scarf with respect to the grain of the lumber and the species of wood. A slope often used for softwood lumber is 1 in 12; this value will develop most of the clear wood strength, and usually the full strength of a piece of structural lumber or plywood containing the normal growth characteristics.

Scarf joints in lumber are usually made by the fabricator when the beam is built, but in plywood are best made at one of the plywood mills equipped for such an operation.

End butt joints may be used in a flange that is laminated from a number of pieces. An end butt joint however should not be considered as transmitting any tensile stress, because it is impossible to glue an end grain joint satisfactorily, or any compressive stress because the joint may be left slightly open in fabrication. Furthermore, butt joints tend to cause points of stress concentration within the flange. Accordingly butt joints should be systematically staggered along the beam—a minimum distance of 24″ between butt joints in the same flange is suggested for $\frac{3}{4}''$ lumber. Also desirable is a maximum amount of staggering of butt joints with respect to the width of the beam. Laminations so jointed should not be considered as contributing to the net moment of inertia of the section although a reasonable number of butt joints does not affect the location of the neutral axis or the stiffness of the beam.

Although several mills are now producing Douglas fir plywood panels up to 40 and 50 feet long, it is likely that stock lengths will be utilized for many built-up beams and girders. For these, web butt joints will usually have to be considered. The common type of butt joint is one with plywood splice plates glued over both sides. The splice plates should be full beam depth where possible, but on at least one side, often both, they cover the web only between the inner surfaces of the flanges. Although some of the flexural stress in the web is transferred into the flange, numerous tests have not shown that any failure resulted from the increased stress in the flange.

Best results occur when the face grain of the splice plates is parallel with the span, particularly if this is also the face grain direction of the web itself. When this is the case, the stress on the secondary glue line between the contacting faces of the web and the splice plate is shear parallel to the grain, a property in which wood is considerably stronger than in rolling shear. However a rolling shear stress still exists within the splice plate between the face and the next inner ply, so that the design stress is not that for shear parallel to the grain. Furthermore plywood constructions are such that there is often a larger percentage of the total thickness parallel to the face grain than perpendicular to it, and in this case the parallel grained veneers are the ones that carry the stress.

The splice plate thickness should be such that the total thickness of the parallel plies in both splice plates equals or exceeds the total thickness of the parallel plies in the web that is being spliced. Also the total thickness of each splice plate should not be less than half the web thickness.

A satisfactory formula for computing the lengths of plywood splice plates which will develop the full bending and shear strength of the web is

$$l=\frac{\sqrt{(Fft/\!/)^2+(vt)^2}}{S_R} \qquad (13)$$

where

l=total length of splice plate measured perpendicular to the joint, inches, when splice plates are glued to both faces of the web.

F=the form factor of the beam (not adjusted by a factor of 10/9ths).

f=allowable fiber stress in bending of the flange lumber, psi.

$t/\!/$=the sum of the thickness of those plies in the web whose grain is parallel to the span, inches.

v=allowable horizontal shear stress in the web, psi.

t=the thickness of the web being spliced, inches.

S_R=allowable rolling shear stress for plywood, psi (not reduced for stress concentration, as in the case of the flange-web joint).

This formula is derived by assuming that the maximum unit stress transmitted across the web joint is equal to the resultant of the maximum fiber stress carried by the parallel plies in the web and of the maximum shear stress carried by the full thickness of the web. Half of this stress is taken into the splice plate on each side of the web by shear in the glue joint, although the flange itself also acts as a splice plate.

One approximation of this formula is that the allowable rolling shear stress is not reduced for stress concentration, although some probably exists. This discrepancy with theory appears justified by numerous test results which show that the values given by this formula are conservative.

As mentioned above the formula is designed to cover both bending and shear simultaneously. Although maximum bending and shear seldom occur at the same section, it is recommended that all splices be sufficient to develop the full web strength so as to insure that the beam will act as a continuous member. A minimum value of 6" is suggested for the splice plate length.

Table I has been prepared as an aid to the designer. It gives splice plate lengths found from Equation 13 for the commonly used plywood thicknesses and lumber stresses. A horizontal shear stress of 210 psi and a rolling shear stress of 79 psi have been assumed, these values corresponding to the EXT-DFPA•A-A grade of plywood commonly used for structural purposes. A form factor of 0.80 has been taken as adequately close, yet conservative. It has also been assumed that the face grain of the plywood webs is parallel to the span.

TABLE I.

SPLICE PLATE LENGTHS, INCHES

$$l=\frac{\sqrt{(Fft/\!/)^2+(vt)^2}}{S_R} \quad *$$

Face grain of webs parallel to span.
v=210 psi
S_R=79 psi

Fiber stress, psi	1200	1400	1600	1800	2000	2200	2400
Thickness							
3/8"-R	6.0"	6.0"	6.0"	6.0"	6.0"	6.0"	6.0"
3/8"-S	6.0"	6.0"	6.0"	6.0"	6.0"	6.0"	6.0"
1/2"	6.0"	6.0"	6.0"	6.0"	6.3"	6.8"	7.5"
5/8"	6.0"	6.0"	6.4"	7.1"	7.8"	8.6"	9.4"
3/4"	6.0"	6.0"	6.4"	7.2"	7.9"	8.6"	9.4"
7/3"	6.6"	7.5"	8.5"	9.5"	10.5"	11.4"	12.4"
1" R	8.0"	9.3"	10.6"	11.8"	13.0"	14.3"	15.5"
1" S	6.0"	6.8"	7.7"	8.6"	9.4"	10.2"	11.2"
1 3/16"	8.3"	9.5"	10.8"	12.0"	13.2"	14.4"	15.7"

*A minimum value of l = 6" has been used.

The spliced section may then be checked to determine the additional stress thrown into the flange from the web. As strain gage measurements have shown that the splice plates do carry stress, a suggested method of analysis is to consider the splice plates, rather than the web, when checking the moment of inertia. (The form factor, however, will remain the same.) A more conservative procedure would be to omit consideration of both splice plates and jointed web in calculating the moment of inertia.

SUMMARY OF DESIGN METHOD

The following outline may be used to summarize the design method. It is assumed that the bending moments and shears have been computed, and that a trial member has been selected for checking. Then the following points should be considered.

1. The maximum bending stress is computed from the formula

$$f = \frac{Mc}{FI} \qquad (2)$$

where F is computed with the gross dimensions of the section and I with the net.

2. The maximum horizontal shear stress is computed from the formula

$$v = \frac{VQ}{It} \qquad (4)$$

where Q and I are computed from the gross dimensions.

3. The maximum flange-web shear stress is found by the formula

$$s_A = \frac{VQ_A}{Id_A} \qquad (6)$$

4. The actual deflection may be checked against the allowable by applying appropriate deflection formulas, taking shear into account.

5. Lateral support should be provided at points depending on the width of the compression flange and the stress existing in it.

6. Bearing and intermediate stiffeners should be provided and spaced so as to develop the full shear strength of the web.

7. Any butt joints occurring in flange members should be properly spaced. Splice plates of appropriate size should be provided for web butt joints.

In the process of design it is usually necessary to adjust the various dimensions of the trial section to make the actual stresses conform with the allowable. In this respect it may be useful to note the effect on the various properties of changes in the cross-sectional dimensions.

For example, if the depth of the beam is increased, the moment of inertia is increased, while the form factor and basic stiffener spacing are decreased.

If the flange depth is increased, the moment of inertia, form factor, and basic stiffener spacing increase, while at the same time the flange-web shear stress is reduced.

When the flange width is increased, the moment of inertia, the flange-web shear stress and the lateral stability increase, and the form factor is reduced.

An increase in the web thickness will cause an increase in the moment of inertia, the form factor and the basic stiffener spacing and a decrease in the horizontal shear stress. However, as the number of webs is decreased, the number of flange-web shear planes is reduced and the flange-web shear stress goes up.

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7. "Design of Plywood Webs in Box Beams." Mimeo No. 1318, and supplements, U. S. Forest Products Laboratory.
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TABLE 2

TOTAL PLY THICKNESSES FOR SELECTED PLYWOOD CONSTRUCTIONS

PLYWOOD THICKNESS	Total Thickness of Parallel* Plies	Total Thickness of Perpendicular* Plies
3/8″ - Rough	.250″	.125″
3/8″ - Sanded	.188″	.188″
1/2″ - R. or S.	.300″	.200″
5/8″ - R. or S.	.375″	.250″
3/4″ - R. or S.	.375″	.375″
7/8″ - R. or S.	.500″	.375″
1″ - Rough	.625″	.375″
1″ - Sanded	.438″	.562″
1 3/16″ - R. or S.	.625″	.562″

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*Refers to direction of face grain.

Example:

It is required to design a girder to support a uniformly distributed permanent load of 1000 #/foot on a 48 foot span. The girder will not be exposed to the weather.

Lumber used in the compression flange will be Dense Select Structural Douglas fir, and in the tension flange Select Structural Douglas fir. Dry unsurfaced lumber is available. Plywood will be unsanded Douglas fir, EXTERIOR type. Allowable stresses are listed below in pounds per square inch.*

	Lumber		Plywood
STRESS	Sel. Struct. Par. 214, W.C.B.L.G.&I.	Dense Sel. Struct. Par. 214a, W.C.B.L.G.&I.	EXT-DFPA•A-A
Ext. fiber in bending	1600	1800	2190
Compression Parallel		1300	1460
Compression Perpendicular	345	380	405
Shear Perpendicular to Plane of Plies			210
Shear Parallel to Plane of Plies			40†
Modulus of Elasticity	1,600,000	1,600,000	1,600,000
Modulus of Rigidity			117,000

(*The Forest Products Laboratory has recently revised its basic stress recommendations for structural lumber. Details as to Douglas fir stresses are available from West Coast Lumbermen's Association, Portland, Oregon.)

†For the design of splice plates an allowable rolling shear stress of 79 psi will be used.

1. Determine Bending Moments and Shears.

Assume weight of beam at 60 #/foot; then shear and moment diagrams will be as shown in Fig. 6.

2. Depth of Trial Section.

Use flexure and deflection formulas simultaneously to solve for depth using approximations suggested on page 3. Let allowable deflection equal L/360.

Flexure — Deflection

$$M=\frac{WL}{8}=\frac{FfI}{c};\qquad d=\frac{(1.25)\ 5WL^3}{384\ EI}=\frac{L}{360}$$

Solving for WL,

$$WL=\frac{8FfI}{h/2}\qquad WL=\frac{384\ EIL}{1.25(360)5(L^2)}$$

Equating and solving for L/h,

$$L/h=\frac{384(1.6\times10^6)}{36000(.8)(1800)}=11.85$$

$$\therefore h=\frac{48\times12}{11.85}=48.5''$$

A trial depth of 48″, corresponding to a stock plywood width, will be used.

3. Web Thickness of Trial Section, from Eq. (1).

$$t=\frac{1.25\ V}{hv}=\frac{1.25(25,440)}{48(210)}=3.16''$$

Select two webs at 1″ plus 1 at $1\frac{3}{16}''$, giving $t=3\frac{3}{16}''$; the center $1\frac{3}{16}''$ web may be dropped off where shear conditions permit, giving a section with two webs at midspan.

4. Trial Flange Dimensions of Center Section.

a. Bending Requirements, from Eq. (2).

$$I=\frac{Mc}{Ff}=\frac{3,660,000(24)}{(.8)(1800)}=61,000\ \text{in.}^4$$

b. Deflection Requirements (assuming constant I) Uniform load deflection formula, increased by 25%.

$$I=\frac{(1.25)(360)5Wl^2}{384E}=\frac{(1.25)(5)(360)(50,880)(48\times12)^2}{384(1.6\times10^6)}$$
$$=\mathbf{61,800}\ \text{in.}^4$$

$$I\ \text{webs}=\frac{2(.625)(48^3)}{12}=11,500\ \text{in.}^4$$

(Table 2 shows that unsanded 1″ plywood has 5⁄8″ of its thickness parallel to the grain of its face plies.)

$$Ifl=61,800-11,500=50,300\ \text{in.}^4$$

Let the flange consist of two pieces, each of horizontally laminated 2″ x 4″s, surfaced during the laminating process to 1⅝″ x 3⅝″, and separated by a $1\frac{3}{16}''$ lumber fill. (An alternate construction would use a $1\frac{3}{16}''$ plywood fill, which would tend to stabilize the flange against checking.) As shown in Fig. 6, Section Three, the flange width is then $8\frac{7}{16}''$ and the maximum distance between inner flange faces required is given by the formula for the moment of inertia.

$$h_1{}^3=48^3-\frac{12I}{w}$$

$$h_1=\sqrt[3]{48^3-\frac{12(50,300)}{8.44}}=33.9''$$

$$\therefore d=\frac{48-33.9}{2}=7.05''$$

Use 5 laminations @ 1⅝″=8⅛″.

5. Flexure Check (Section 3).

$$I=\frac{2(.625)(48^3)}{12}+\frac{8\frac{7}{16}(48^3-31\frac{3}{4}{}^3)}{12}$$

$$=11,500+55,400=\mathbf{66,900}\ \text{in.}^4$$

$$\left.\begin{array}{l} d/h=8\frac{1}{8}/48=.169 \\ t_1/t_2=2/10\frac{7}{16}=.192 \\ F=\dfrac{.723(10)}{(9)}=\mathbf{.804} \end{array}\right\}\ \text{See Section 7 Page 3}$$

The factor of 10/9ths applies to the working stresses, but is considered with the form factor for convenience.

$$\text{Allowable Resisting Moment} = \frac{FfI}{c} =$$

$$\frac{(.804)(1800)(66{,}900)}{24} = \mathbf{4{,}030{,}000}\ \text{in.-lbs.}$$

(It may be noted that the tension flange stress will be within the allowable wherever the compression flange stress is, because $F = .723 < \frac{1600}{1800} = 0.89$.)

6. Horizontal Shear Check (Section 3).

$$Q = 2(.625)(24)(12) + 8\tfrac{7}{16}(8\tfrac{1}{8})(19\tfrac{15}{16})$$

$$= 360 + 1365 = 1{,}725\ \text{in.}^3$$

$$V = \frac{vIt}{Q} = \frac{210(66{,}900)(2)}{1725} = \mathbf{16{,}280}\ \text{lbs.} \qquad (4)$$

7. Flange-web Shear Check (Section 3).

$$V = \frac{sId}{Q\,fl}\ \frac{(\Sigma t)}{(t_1)} = \frac{40\ (66{,}900(8\tfrac{1}{8})}{1365}\ \frac{(2)}{(1)}$$

$$= 31{,}800\ \text{lbs.} \qquad (8)$$

∴ Allowable shear on center section is 16,280 lbs., governed by horizontal shear at the neutral axis.

8. Other Sections Through Beam.

The innermost flange laminations may be dropped off, as stress conditions permit, from the center toward the reactions. Sections shown in Fig. 6 have been checked by the methods shown for Section 3 to determine resisting moments and shear.

Section 2.

$I = 59{,}100$ in.^{4}

$d/h = .135$

$t_1/t_2 = .192$

$F = \frac{.706}{.9} = .785$

Resist. Mom. = 3,480,000 in.-lbs.

Resist. Hor. Shear = **16,600** lbs.

Resist. Fl.-web Shear = 26,800 lbs.

Section 2a.

$I_{net} = 52{,}500$

$d/h = .135$

$t_1/t_2 = .192$

$F = \frac{.706}{.9} = .785$

Res. Mom = 3,090,000 in.-lbs.

The net moment of inertia of Section 2a is computed ignoring the center web because of the butt joint.

Section 1.

$I = 50{,}350$ in.^{4}

$d/h = .1015$

$t_1/t_2 = .305$

$F = \frac{.735}{.9} = .817$

Res. Mom. = 3,080,000 in.-lbs.

Res. Hor. Shear = **25,900** lbs.

Res. Fl.-web Shear = 40,500 lbs.

The transition from Section 3 to Section 2 may be made where the bending moment is 3,480,000 in.-lbs., equal to the resisting moment of Section 2. This point is found to be 18½ feet from the reaction. The transition will be made gradually by scarfing the excess lamination at a slope of 1 to 12.

The change from two to three webs may be made at a point where the shear is 16,600 lbs., the allowable horizontal shear on Section 2. This point is found from the shear diagram to be 8.35 feet from the reaction; because of the presence of secondary stresses at the point of discontinuity, the change will be made 9.0 feet from the reaction, requiring either the use of a 9 foot plywood panel or a splice.

9. Check Deflection.

Because the beam sections vary along its length, the section properties used in the deflection formulas should be an average weighted with respect to the area of the shear diagram and the moment about the reaction of the area of the moment diagrams. In this case it should be sufficiently accurate to consider the half-span to consist of three parts of constant dimensions, each 8 feet long.

a. Bending Deflection.

For use in the deflection formula an average I for a symmetrical beam symmetrically loaded may be found from the formula

$$I\ \text{avg.} = \frac{Ax}{\frac{Ax}{I}}$$

where

A = the area of a section of the moment diagram.

x = the distance from the near reaction to the center of gravity of the area under consideration.

I = the moment of inertia of that portion of the beam corresponding to the area under consideration.

Calculations (not shown here) disclose that the moment diagram A and x have relative values as shown in the table below.

Part	A	x	I	Ax	$\frac{Ax}{I}$
1	1.356	5	50,350	6,780	.135
2	2.035	12	59,100	24,420	.414
3	.810	13	59,100	10,530	.178
4	3.250	20	66,900	65,000	.973
5	.273	21	66,900	5,730	.086
				112,460	1.786

$$I \text{ avg.} = \frac{112{,}460}{1.786} = 63{,}000 \text{ in.}^4$$

$$d_b = \frac{5(50{,}880)(48 \times 12)^3}{384(1.10 \times 1{,}600{,}000)(63{,}000)} = 1.140''$$

(See Item 4, p. 7.)

b. Shear Deflection.

Calculations using the shear diagram and Fig. 3 may be made.

Section	A	d/h	t_1/t_2	K	AK	I	AI
1	2.120	.1015	.305	.50	1.060	50,350	106,700
2	1.272	.135	.192	.79	.992	59,100	75,300
3	.424	.169	.192	.84	.356	66,900	28,300
	3.816				2.408		210,300

$$K = \frac{2.408}{3.816} = \mathbf{.63}$$

$$I = \frac{210{,}300}{3.816} = \mathbf{55{,}200} \text{ in.}^4$$

$$d_s = \frac{PlKh^2C}{GI} = \frac{(50{,}880)(48 \times 12)(.63)(48)^2(.05)}{(117{,}000)(55{,}200)} = 0.329'' \qquad (10)$$

Total deflection $= 1.140 + 0.329 = \mathbf{1.47''}$

$$\text{Allowable} = \frac{48 \times 12}{360} = 1.60''$$

10. Design Splices.

All end joints in the flanges, except the junction of the center fill with the web at the sixth-points, will be scarfed at a slope of 1 in 12 and glued. The joints in the 1″ plywood webs will be butted and covered both sides with ½″ plywood splice plates. The length of the splice plates is computed from Equation 13.

$$l = \frac{\sqrt{(Fft/\!/)^2 + (vt)^2}}{S_R} = \frac{\sqrt{(.74)(1800)(.625)^2 + (210)(1)^2}}{79} = 10.8''$$

(The length could have been taken from Table I with sufficient accuracy as 11.7″.)

Cutting economy, with a standard plywood panel length, indicates that a splice plate length of 12″ may as well be used.

The added stress thrown into the flange around the web splice will be computed by considering the moment of inertia of the splice plates as replacing that of the web.

It is assumed that 8′ long panels will be used (except for the center web, which will be a 9′ panel) thus requiring splices at the sixth, third and center points of the span. Comparison of the resisting moments with the bending moments shows the most critical splice section occurs at the third-point, in Section 2b.

$$I\text{spl. pl.} = \frac{2(.3)(48^3 + 35^3)}{12} = 7670 \text{ in.}^4$$

(See Table 2.)

$$I\text{fl.} = \frac{8\tfrac{7}{16}(48^3 - 35^3)}{12} = 47{,}600 \text{ in.}^4$$

$$I = 7{,}670 + 47{,}600 = 55{,}270 \text{ in.}^4$$

$$\text{Resis. Mom.} = \frac{55{,}270(3{,}480{,}000)}{59{,}100} = 3{,}250{,}000 \text{ in.-lbs.}$$

(See values for Section 2.)

Bending Mom. $= 3{,}250{,}000$ in.-lbs.

11. Stiffener Design.

a. Bearing Stiffeners

The required thickness of stiffeners at the reactions is

$$x = \frac{V}{wc\perp} = \frac{25{,}400}{(10\tfrac{7}{16})(345)} = 7.06'' \qquad (11)$$

Then the bearing stiffeners will be cut from two 4″ x 8″s. No other bearing stiffeners are required.

b. Intermediate Stiffeners

Intermediate stiffeners will be made 1⅝″ thick, and of a width equal to that of the flange. However, at the point of discontinuity of the center web—i.e., 9′ from the reaction—3⅝″ x 3⅝″ stiffeners will be used to help distribute the secondary stresses.

Stiffener spacings will be those for 1″ plywood, the minimum used in this example. Fig 4 shows that for a clear distance between flanges of 38¼″ (See Section 1, Fig. 6) the basic stiffener spacing b=48″. Since the maximum actual shear stress at the reaction is practically 100% of the allowable, the maximum spacing of 48″ will be used. Then as the thickness of the end bearing stiffener is 7½″ and of the intermediate stiffener 1⅝″, the first intermediate stiffener could be centered $48'' + 7\tfrac{1}{2}'' + \tfrac{13}{16}'' = 56\tfrac{5}{16}''$ from the end. Reference to the shear diagram shows that at this point the external shear will be 20,500 lbs., or

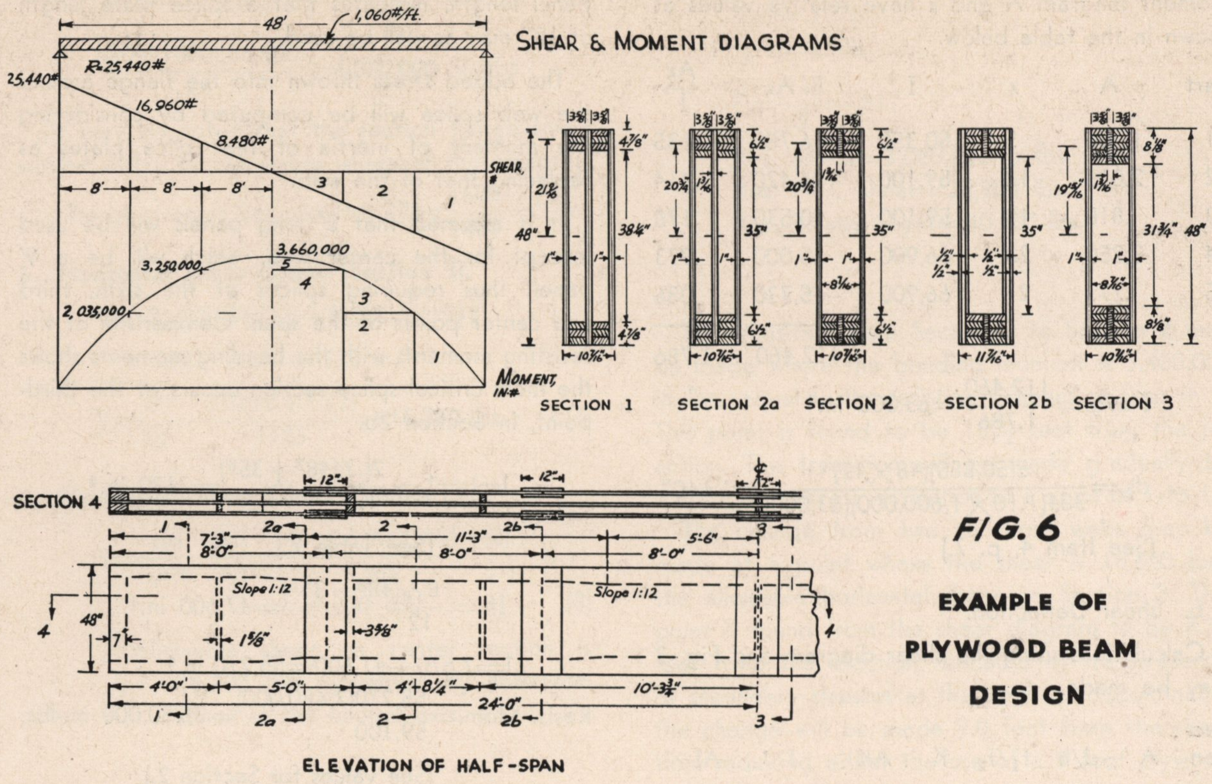

FIG. 6

EXAMPLE OF PLYWOOD BEAM DESIGN

79% of the resisting horizontal shear strength of the section. Then the allowable clear distance between the first and second stiffeners will be

$$b' = 48\left(1 + \frac{100-79}{25}\right) = 88.4''. \qquad (12)$$

The second intermediate stiffener will be located where the center web ends, 9′ from the reaction, and the first stiffener will be located 48″ from the reaction.

The external shear at the second stiffener is found to be 15,900 lbs., which is 96% of the allowable on the section. The basic spacing for a flange distance of 35″ is 48½″ and the maximum allowable clear distance between the second and third stiffener is

$$b' = 48\tfrac{1}{2}\left(1 + \frac{100-96}{25}\right) = 56\tfrac{1}{4}''.$$

The external shear at the third stiffener is 10,800 lbs., or 65% of the allowable, and the permissible spacing between the third and fourth stiffener is

$$b' = 48\tfrac{1}{2}\left(1 + \frac{100-65}{25}\right) = 116\tfrac{1}{2}''.$$

It will be sufficient to place the fourth stiffener at the centerline, a distance of 10′ 1 5/16″ from the third stiffener.

12. Lateral Stability Check.

Because the load is uniformly distributed, presumably the top flange is supported laterally at frequent intervals. However, if this were not the case, the spacing of supports would depend on the percentage of allowable stress in the compression flange.

In Part 3, the percentage of allowable stress is

$$\frac{3,660,000}{4,030,000} = 91\%.$$

In Parts 2 and 1, these percentages are found to be 96% and 48% respectively.

As the modulus of elasticity of the flange lumber is 1,600,000 psi, Fig. 5 may be used with a value of C=1300 psi, the allowable compression stress. For 91%, 96% and 48% stresses, there are found respectively l/w values of 16.3, 13.5 and 26.4. The full beam width of 10 7/16″ may be taken for w, so that corresponding distances between lateral supports are 14.2′, 11.8′ and 23.0′. One arrangement of lateral supports that will meet these requirements could be achieved by placing them at the ends, quarter-points and midspan.

A check on the actual dimensions of the girder shows the weight to be 58.5 lbs. per linear foot, sufficiently close to the original estimate.

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